FINAL FOUNDATION REPORT

Christina River Bridge

City of Wilmington, Delaware

Contract No. 25-121-02

Prepared for:

Delaware Department of Transportation

RK&K Commission No. 104-130-03G

January 29, 2016

RUMMEL, KLEPPER & KAHL, LLP
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Baltimore, Maryland 21217

Final Foundation Report
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Prepared for:
Delaware Department of Transportation
800 Bay Road
Dover, Delaware 19903

Rummel, Klepper & Kahl, LLP, is pleased to submit the Final Foundation Report (FFR) for the Christina River Bridge Project.

The FFR describes the subsurface exploration program, the general site conditions, proposed construction, the subsurface conditions, and presents geotechnical engineering data for this project.

This FFR supersedes in its entirety the Final Foundation Report (FFR) dated July 30, 2015 prepared by RK&K. This report also addresses comments provided by AECOM dated September 28, 2015. Our responses to these comments will be included in a separate submission.

We appreciate having had the opportunity to provide geotechnical consultation for this project.

Very truly yours,
RUMMEL, KLEPPER & KAHL, LLP

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1 INTRODUCTION

In accordance with our proposal dated May 14, 2008, Rummel, Klepper & Kahl, LLP (RK&K) has completed the Final Foundation Report for the Christina River Bridge project located in City of Wilmington, Delaware.

The purpose of this study was to determine general subsurface conditions at the project site and to evaluate those conditions with respect to geotechnical engineering considerations for the proposed construction. The specific scope of our services on this project consisted of exploring the subsurface conditions using soil borings, rock coring, performing laboratory testing of selected soil and rock samples, performing Cone Penetration Test (CPT), performing in-situ Pressuremeter Test (PMT) in soil and in rock, evaluating the subsurface conditions encountered in the test borings, developing geotechnical recommendations to aid design, and submitting our findings in a report. Based on this geotechnical study, recommendations are provided for the proposed bridge foundations, bridge wingwall foundation design, retaining walls, approach embankments, pedestrian underpass, flexible pavement and other geotechnical concerns.

Also included in this report are descriptions of the field and laboratory testing on which this report is based. The results of this work are contained in the appendix of this report.

2 SITE AND PROJECT DESCRIPTION

2.1 SITE DESCRIPTION

The project site is located south of the Wilmington Riverfront Shipyard shops and west of the intersection of South Walnut Street and James Court in the City of Wilmington, Delaware as shown in Figure A-1. The proposed site is developed and spans the banks of the Christina River. The west bank of the river along the proposed site consists of open areas with the Wilmington Riverfront Shipyard Shops, a one-story shopping strip mall, and parking lot located north of the proposed bridge. The east bank of the river is an industrial park consisting of open areas with multiple commercial stores, light industrial buildings, and warehouses located on the north and south of the proposed alignment.

The existing ground surface on the west bank of the river is approximately level with the ground surface elevation at approximately EL 11. A drainage swale runs parallel to the river along the west bank. The bottom of the swale is located at approximately EL 4. The existing ground surface on the east bank of the river slopes gently up from west to east and the existing ground surface elevation ranges from approximately EL 4 to EL 8. Table 2.1 summarizes the water elevation and the estimated flood elevations.

Table 2.1 – Summary of Flood Elevation					
River Station	**Spring High Tide**	**100-yr Flood**		**500-yr Flood**	
1154	EL+3.59	Spring High Tide	EL+4.53	Spring High Tide	EL+5.62
		Peak Tide	EL +9.00	Peak Tide	EL 10.75
Scour elevation obtained from the “FINAL HYDROLOGIC AND HYDRAULIC ANALYSIS REPORT FOR THE CHRISTINA RIVER BRIDGE” dated July 2015					

According to the Dravo plans from 1943, a meandering stream flowed along the proposed road alignment on the west bank into the Christina River. The Dravo plans show a basin immediately to the west of the proposed bridge alignment and a canal to the south. The proposed Abutment A on the west bank is located within the historic stream. The location of the historic shoreline, stream, basin, canal and the proposed alignment are shown in Figure A-4.

2.2 PROJECT DESCRIPTION

The proposed construction will consist of a three span bridge and associated ramps spanning the Christina River Bridge. The width of the proposed bridge will be approximately 45-ft and will consist of two travel lanes and a pedestrian path. The west abutment (Abutment A) will be at approximately STA 436+01 and the east abutment (Abutment B) will be at approximately STA 440+70. The bridge piers will be located at approximately STA 437+45 and STA 439+25. The span length of the bridge is summarized in Table 2.2

Table 2.2 – Span Lengths

Span Number	Location	Length (ft)
1	Abutment A to Pier 1	145
2	Pier 1 to Pier 2	180
3	Pier 2 to Abutment B	145

The proposed grade at Abutment A will be at approximately EL 21.5 and will slope downward to meet existing grade near STA 432+66 near EL 11. The proposed grade at Abutment B will be at approximately EL 19.6 and will slope downward to approximately EL +9.5. The existing grade east of Abutment B varies from approximately EL+4 to EL +7. The finished grade for the proposed roadway embankment will be at approximately EL +9.5 at STA 444+85 with a 0.5% upward gradient to approximately EL 11.5 at STA 448+90. . The width of the west side approach ramp will be approximately 45-ft wide. The approach ramp on the east side will be approximately 45-ft wide at the abutment and flares to approximately 55-ft at STA 443+00.

The structural loads per individual drilled shaft for the foundation design of the proposed abutments and piers are summarized in Table 2.3. The structural loads for each substructure foundation cap is summarized in Table 2.4.

The weight of the backfill between the wing walls and the abutment walls will be transferred to the wing wall and abutment foundations using a structural slab.

Table 2.3 – Summary of Abutment and Pier Axial Loads					
Structure Unit	**Bottom of Footing**	**100-Year Scour Elevation**	**500-Year Scour Elevation**	**Factored Load (kips)**	**Required Nominal Resistance (kips)**
Abutment A	EL +1.0	+4.0	+4.0	1107	1582
Pier 1	EL -4.55	-26.9	-28.9	2263	3233
Pier 2	EL -4.55	-18.0	-20.9	2240	3200
Abutment B	EL 0.0	+1.0	-5.0	924	1319
Factored and Nominal Resistances are for individual drilled shaft. Nominal Resistances are based on a Resistance Factor $Ø_{stat}$ = 0.7 assuming a SLT Scour elevation obtained from the “FINAL HYDROLOGIC AND HYDRAULIC ANALYSIS REPORT FOR THE CHRISTINA RIVER BRIDGE” dated July 2015					

Table 2.4 – Summary of Abutment and Pier Foundation Loads						
Structure Unit	**Limit State**	**Vertical Load, Fy (kips)**	**Shear Load**		**Moments About**	
			Longitudinal, Fz (kips)	**Transverse, Fx (kips)**	**Longitudinal, Mz (ft-kips)**	**Transverse, Mx (ft-kips)**
Abutment A	Service	2831	66	47	1368	2111
	Strength	3628	55	59	2195	1712
Pier 1	Service	4560	76	73	3427	2190
	Strength	5881	109	144	6240	2880
Pier 2	Service	4535	130	103	3838	2619
	Strength	5849	145	156	6480	2850
Abutment B	Service	2339	130.3	72.7	1511.8	3197.8
	Strength	3019	108.3	78.3	2246.2	4610.2

The proposed construction will also include construction of the riverwalk behind Abutment A on the west bank of the river. The river walk will consist of an approximately 31-ft wide underpass west of Abutment A near STA 435+59. The underpass will consist of cast-in-place concrete arch section. The bottom of footing for the arch will be located at approximately EL +1.0 on the east side and EL +4 on the west side. The construction will also consist of a set of stairs from the Riverwalk to the pedestrian pathway on the bridge. The bottom of footing for the stair grade-beams will be located at approximately EL +8. The foundation loads for the underpass and the stairs are summarized in Table 2.5.

Table 2.5 – Summary of Underpass and Stairs Foundation Loads

Structure Unit	Limit State	Vertical Load, Fy (kips)	Shear Load Longitudinal, Fz (kips)	Moments About Transverse, Mx (ft-kips)
Underpass East Wall	Service	2073	47	538
	Strength	3006	70	80
Underpass West Wall	Service	1200	7	82
	Strength	1817	11	124
Stairs	Service	182	26	211
	Strength	229	33	265

3 FIELD AND LABORATORY WORK

3.1 FIELD EXPLORATION

The subsurface exploration consisted of drilling Standard Penetration Test (SPT) borings, Pressuremeter Tests (PMT) in soil and rock, Cone Penetration Test (CPT) probes, and performing laboratory testing on representative samples. The subsurface exploration was performed in two phases.

Phase I

The first phase of subsurface exploration consisted of drilling 32 Standard Penetration Test (SPT) borings, performing 10 soil Pressuremeter Tests (PMT), 5 rock PMTs, and performing laboratory testing on representative samples. Table 3.1 summarizes the borings drilled for the proposed structures.

Table 3.1 – Summary of Phase I Subsurface Exploration Program

Structure	SPT Borings
Abutment A	AA-1, AA-2, SA-1, SA-1A, SA-2
Pier 1	P1-1, P1-2, P1-2A
Pier 2	P2-1, P2-2
Abutment B	AB-1, AB-2, AB-3, AB-4
Retaining Wall (West Side)	RW-1, RW-2, RW-2A, RW-3, RW-4
Retaining Wall (East Side)	RW-5, RW-6, RW-6A, RW-7, RW-8
Roadway	R-7, R-8, R-9, R-10, R-11
Walkway	W-1, W-2, W-4

All land borings were performed with a CME 55 ATV mounted drill rig except borings RW-3 and RW-4 where a truck mounted drill rig was used. All water borings were performed with a CME 45 skid rig from a barge. All the drilling was performed from September 8, 2011 to January 5, 2012 by Walton Corporation of Newark Delaware under contract to DelDOT. The borings were drilled at the approximate location of the proposed abutments, piers, retaining walls, walkway and roadway. Elevations were determined by survey crews. Borings locations are shown in Figures A-2a through A-2e.

The borings for the abutments extended to depths ranging from 116.5-ft to 145.5-ft below the existing ground surface and the pier borings extended to depths ranging from 73-ft to 129.0-ft below the mudline. The retaining wall borings were drilled to depths ranging from 73.7-ft to 130.0-ft below the existing ground surface. The walkway borings were drilled to 60-ft below the mudline. All roadway borings were drilled to a depth of 10-ft below the existing ground surface.

Phase II

The second phase of subsurface exploration consisted of drilling 13 SPT borings, 3 offset borings to obtain Shelby tube samples, 5 Piezocones (CPTU) probes, and performing laboratory testing on representative soil samples. Double ring infiltration tests were performed at the three stormwater management boring locations. Table 3.2 summarizes the borings drilled.

Table 3.2 – Summary of Phase II Subsurface Exploration Program

Structure	SPT Borings	CPT Probes
Abutment A	SA-3	CPT-2
Abutment B	AB-5	CPT-3
Retaining Wall (West Side)	RW-9	CPT-1
Retaining Wall (East Side)		CPT-4, CPT-5
Roadway	R-12, R-13, R-14, R-15, R-16, R-17, R-18	
Stormwater Management	SWM-1, SWM-2, SWM-3	

All borings were performed with a CME 55 ATV mounted drill rig using a safety hammer. All the drilling was performed from September 3, 2013 to September 13, 2013 by Walton Corporation of Newark Delaware under contract to DelDOT. The borings were drilled at the approximate location of the proposed abutments, retaining walls, roadway, and storm water management ponds. Borings locations are shown in Figures A-2a through A-2e. The CPTU soundings were performed by Geo-Technology Associates, Inc. (GTA) of Newark, Delaware under contract to DelDOT.

The structure soil borings extended to depths ranging from 50.0-ft to 100.0-ft below the existing ground surface. All stormwater management borings were drilled to a depth of 16.0-ft below the existing ground surface. All roadway borings were drilled to a depth of 20-ft below the existing ground surface.

3.2 SOIL SAMPLING

The land borings were advanced with hollow stem augers to depths ranging from 50 to 60-ft and to the termination depths using mud rotary drilling. The soil samples were obtained at a maximum 5-ft interval in accordance with the SPT method using a safety hammer. In general, the SPT consists of advancing a 2-inch outside diameter sampling spoon 18-inches by driving it with a 140-pound hammer falling 30-inches. The values reported on the boring logs are the blows required to advance three successive increments. The first 6-inch increment is considered as seating. The sum of the number of blows for the second and third increments is the "N" value, which is an index of soil strength.

Relatively undisturbed soil samples were obtained using Shelby tubes. The Shelby tube consists of a thin-walled steel tube 76 mm in diameter. These tubes were hydraulically pressed into fine-grained soils to retrieve an undisturbed soil sample for soil strength and consolidation testing.

In addition, five bulk bag samples were obtained from auger cuttings from borings R-7 through R-11.

The soils were classified in general accordance with the Unified Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials (AASHTO). The AASHTO graphical and letter symbols are shown on the Summary of Boring Data, Figure A-3a through A-3g. A RK&K field engineer recorded the classifications, observations, water and cave in depths and field sampling information on the Test Boring Logs contained in Appendix B. Descriptions of the soils classification systems and sample procedures are also included in Appendix B.

3.3 ROCK SAMPLING

Bedrock was sampled using an NW/L2 diamond bit with a double tube, swivel type barrel, which provides a 2.0-inch diameter core. The core description, core recovery, the Rock Quality Designation (RQD), and other pertinent information were recorded on the Test Boring Logs, the

Rock Core Description Sheet, and on the Summary of Boring Data. The RQD value reflects the quality and fracture spacing of the rock and is defined as the sum of the length of rock pieces greater than 4-inches divided by the total core run length. The percentage of core recovery and RQD values provide an understanding of the physical and engineering properties of the rock. The fracture frequency of the rock cores are indicated on the boring logs located in Appendix B. Fracture frequency is the total number of natural fractures occurring in a foot of rock core recovered.

Descriptions of the rock classification system and sampling procedures are also included in Appendix B.

3.4 PRESSUREMETER TEST

In situ testing included pressuremeter tests (PMT) in soil and in rock. The boreholes were advanced with an ATV mounted drill rig or a skid rig mounted on a barge. The pressuremeter tests in soil were performed in general accordance with ASTM D4719 – Standard Test Method for Pressuremeter Testing in Soil. The test consisted of placing the probe in a pre-drilled hole and expanding the probe while measuring changes in volume for each equal pressure increment.

The probes were calibrated in air and in NX-sized, steel casing to estimate the stiffness of the measuring cell and the expansion of the system. The test holes were created by advancing a 3-inch tricone bit below the hollow stem augers in soil and using a NW/L2 diamond bit in rock. The probe was then positioned at the testing elevation in the hole with the drill string. The probe was pressurized in equal increments until the injected volume of water neared probe capacity or until the probe ruptured. At each pressure increment, the volume of injected fluid or the radial expansion of the probe was recorded at 30 and 60 second intervals. In some cases, load-unload cycles and creep tests were performed.

PMT tests were performed by In-situ Soil Testing, Inc of Lancaster Virginia. Table 3.3 and 3.4 summarizes the PMT test results performed in soil and rock, respectively. The results of the PMT testing are contained within Appendix C of this report. The strata are described in Section 4.2 of this report.

Table 3.3 – Summary of Pressuremeter Test Result in Soil								
Boring No.	**Depth**	**Elevation**	**Stratum**	**Limiting Pressure (tsf)**	**Initial Modulus (tsf)**	**Average Reload Modulus (tsf)**	**Average Unload Modulus (tsf)**	**PL/E_0**
AA-1	58.60	-54.83	II	22.45	316.41	1056.28	1532.99	14.1
	81.30	-77.53	III	50.13	798.87	2600.76	**	17.3
P1-1	32.00	-53.05	II	22.97	288.22	976.39	1714.17	13.7
	62.40	-83.45	III	51.17	813.49	2300.53	4665.29	15.9
P2-2	33.00	-49.90	II	18.80	303.88	727.34	1302.21	18.2
	70.00	-86.90	III	40.73	644.32	2065.05	4115.48	17.4
AB-1	63.00	-57.00	II	27.15	439.64	1481.30	**	17.8
	88.10	-82.10	III	27.15	851.08	3068.07	**	36.5
AB-4	67.10	-58.87	II	18.80	352.96	942.98	**	18.8
	100.90	-92.67	III	54.30	889.72	2720.85	6674.46	17.7
** Unload Cycle was not performed								

Table 3.4 – Summary of Pressuremeter Test Result in Rock			
Boring No.	**Depth**	**Elevation**	**Tangent Elastic Modulus (tsf)**
AA-1	130.00	-126.23	30,637
P1-1	108.50	-129.55	692
P2-2	112.00	-128.90	3,764
AB-4	114.00	-105.77	1,673

3.5 CONE PENETRATION TEST

Geo-Technology Associates, Inc. (GTA) conducted five Cone Penetration Test soundings (CPTU), with a track mounted CPT rig at the proposed ramp locations on the east and west bank of the river as shown in Figures A-2a through A-2e. The CPTU probes were performed on September 11, 2013.

The CPTU (ASTM D5778) consists of pushing a series of cylindrical rods with a cone at the base into the soil at a constant rate. Continuous measurement of penetration resistance on the cone tip (Qc) and friction on a friction sleeve (Fs) were recorded during the penetration. Pore pressures were measured using a pressure transducer that measures the pore water pressure generated during penetration. The piezometer to measure the pore water pressure was located at the u2 location behind the collar of the cone tip. Correlations have been developed to estimate the soil types, friction angle, undrained shear strength, modulus, stress history, and SPT N-value from the measured data. The results of the CPTU probe are contained within Appendix C of this report.

3.6 GROUNDWATER

Where possible depth to groundwater was noted during the drilling operations and groundwater levels were measured at the completion of drilling. The depth to the bottom of each borehole was also measured after the removal of the drilling augers to determine the susceptibility of the borehole to collapse or cave. Where rock was sampled using rock coring techniques, it was not possible to obtain meaningful water level readings upon completion of the borings.

It is generally desirable to allow test borings to remain open for at least 24 hours after the completion of drilling and the removal of the drill tools and casing from the borehole. The purpose of this procedure is to allow the groundwater level in each borehole to recover from the effects of the test drilling. In clay soils, the length of time may extend several days before the groundwater level recovers to the pre-drilling elevation.

It was necessary to backfill the borings immediately after the completion of drilling to provide safe conditions because the borings were located in areas frequented by pedestrians. In cases where the boring was immediately backfilled with a tremie grout, the boring logs note the depth where groundwater was observed either within the recovered soil sample, on the split barrel samples, on the drill rods, or in the soil brought to the surface by the hollow stem augers.

3.7 LABORATORY TESTING

All laboratory testing was conducted by Material and Research (M&R) section of DelDOT and Geotechnical Associates (GTA) of New Castle, Delaware. The laboratory testing for this project consist of determining the natural moisture content, the grain-size distribution and the Atterberg limits for selected samples. Results of the classification testing are summarized in Table C-1 in Appendix C. Natural Moisture Content test results are shown on the Test Boring Logs in Appendix B. Detailed test results including Grain-size distribution graphs are included in Appendix C. Description of the strata are in Section 4 of this Report.

Laboratory testing to determine the stress history, consolidation parameters, shear strength and modulus for the Shelby tube samples consisted of consolidation tests, Unconsolidated Undrained Triaxial Tests (UU), and Consolidated Undrained Triaxial Tests (CU). Phase I shear strength testing and consolidation testing of Shelby tube samples are summarized in Tables 3.5 and 3.6, respectively, and detailed results are contained in Appendix C. Preconsolidation pressures indicated in Table 3.6 are obtained from the consolidation curves on the laboratory test results contained in Appendix C using the Casagrande and Work-Energy methods. Phase II laboratory testing is in process and will be updated in the Final Geotechnical Engineering Report.

Table 3.5 – Summary of Shear Strength Testing								
Boring No. / Sample No.	Test	Depth (ft)	Stratum	Su (psf)	Strength Parameters			
					c (psf)	Phi	c' (psf)	Phi'
AA-1 / U-1	CIUC	21.0 - 23.0	Ia	--	39	17.6	166	36.4
AB-2 / U-1	CIUC	12.0 - 14.0	Ia	--	171	13.7	0	35.9
AB-3 / U-2	CIUC	20.0 - 22.0	Ia	--	0	22.3	0	38.2
AB-4 / U-1	CIUC	14.0 - 16.0	Ia	--	0	20.1	36	36.2
AB-4 / U-2	CIUC	28.0 - 30.0	Ia	--	0	20.5	0	37.8
RW-1 / U-1	CIUC	14.0 - 16.0	Ia	--	229	13.3	0	43.9
RW-1 / U-2	CIUC	23.0 - 25.0	Ia	--	0	17.9	0	33.8
RW-2 / U-1	CIUC	18.0 - 20.0	Ia	--	0	17	127	39.4
RW-3 / U-2	CIUC	20.0 - 22.0	Ia	--	132	18.7	0	41.5
RW-5 / U-2	CIUC	30.0 - 32.0	Ia	--	111	18.7	186	32.9
RW-6 / U-2	CIUC	20.0 - 22.0	Ia	--	86	16.5	0	34.3
RW-8 / U-1	CIUC	8.0 - 10.0	Ia	--	115	10.8	94	18.2
RW-8 / U-2	CIUC	18.0 - 20.0	Ia	--	539	8.7	569	12.1
W-1 / U-1	CIUC	12.0 - 14.0	Ia	--	45	19.6	0	37.3
RW-4 / U-1	UU	14.0 - 16.0	Ia	441	--	--	--	--
RW-5 / U-1	UU	16.0 - 18.0	Ia	68	--	--	--	--
RW-6 / U-1	UU	12.0 - 14.0	Ia	29	--	--	--	--
RW-7 / U-1	UU	10.0 - 12.0	Ia	123	--	--	--	--
SA-1 / U-1	UU	14.0 - 16.0	Ia	603	--	--	--	--
SA-2 / U-1	UU	18.0 - 20.0	Ia	701	--	--	--	--
W-4 / U-1	UU	51.0 - 52.5	II	**197	** Sample may be disturbed			

c: Cohesion
c': Drained Cohesion

ϕ: Friction Angle
ϕ': Drained Friction Angle

UU: Unconsolidated Undrained Triaxial Test
CIUC: Consolidated Isotropic Undrained Triaxial Test

Table 3.6 – Summary of Consolidation Testing

Boring No. / Sample No.	Depth (ft)	Stratum	NMC	P_o (tsf)	P_c (tsf)	OCR	Dry Unit Wt. (pcf)	CC	CR
AA-1 / U-1	21	Ia	75.7	1.32	0.55	0.42	55.3	0.172	0.028
AA-2 / U-1	12	Ia	156.0	0.72	0.55	0.76	30.6	0.245	0.027
SA-1 U-1	14	Ia	63.7	0.68	0.86	1.26	64.1	0.184	0.033
SA-2 / U-1	18	Ia	101.0	0.77	0.74	0.96	44.2	0.236	0.036
AB-1 / U-1	12	Ia	68.8	0.72	0.64	0.89	60.1	0.144	0.019
AB-2 / U-1	12	Ia	50.7	0.55	0.82	1.49	72.2	0.146	0.013
AB-3 / U-2	20	Ia	51.6	1.26	0.52	0.41	73.5	0.136	0.009
AB-4 / U-1	14	Ia	49.4	0.65	1.37	2.11	72.6	0.151	0.012
RW-1 / U-1	14	Ia	130.1	0.78	0.87	1.12	36.2	0.226	0.041
RW-1 / U-2	23	Ia	22.0	1.03	2.19	2.13	108.7	0.065	0.006
RW-2 / U-1	18	Ia	133.0	1.14	0.7	0.61	36.8	0.239	0.043
RW-3 / U-1	12	Ia	52.3	0.78	1.13	1.45	72.2	0.160	0.022
RW-3 / U-2	20	Ia	53.3	1.04	0.8	0.77	59.1	0.210	0.030
RW-4 / U-1	14	Ia	66.2	0.68	1.09	1.60	60.8	0.172	0.024
RW-4 / U-2	20	Ia	87.7	0.85	0.73	0.86	49.6	0.233	0.036
RW-5 / U-1	16	Ia	59.2	0.71	1.4	1.97	64.8	0.156	0.016
RW-5 / U-2	30	Ia	69.5	1.11	0.52	0.47	59	0.145	0.022
RW-6 / U-1	12	Ia	62.0	0.44	0.62	1.41	62.6	0.160	0.011
RW-7 / U-1	10	Ia	81.9	0.5	0.74	1.48	52.4	0.199	0.026
RW-8 / U-1	8	Ia	71.7	0.54	0.51	0.94	57.6	0.186	0.018
RW-8 / U-2	18	Ia	89.2	0.67	0.49	0.73	48.7	0.207	0.028
W-1 / U-1	12	Ia	87.7	0.78	0.87	1.12	49.4	0.211	0.029
W-4 / U-1	51	II	34.0	1.5	0.62	0.41	92	0.105	0.039

P_o: Effective Overburden
P_c: Preconsolidation Pressure
OCR: Overconsolidation Pressure
NMC: Natural Moisture Content
CC: Compression Ratio ($Cc/1+e_o$)
CR: Recompression Ratio ($Cr/1+e_o$)

Corrosion Potential testing was performed on select soil samples from the land borings. The testing consisted of determining the Resistivity, pH, Redox Potential, and Sulfides for the soil samples. Results of the Corrosion Potential testing are summarized in Table 3.7

Table 3.7 – Summary of Corrosion Potential Testing

Boring No.	Depth (ft)	Resistivity (ohm-cm)	pH	Redox Potential (mV)	Sulfides
AA-1	14.0 – 16.0	470	5.2	200.0	Negative
AA-1	53.0 – 55.0	1500	5.8	260.0	Negative
SA-1	65.0 -67.0	1000	6.1	278.0	Negative
AB-1	53.0 -57.0	1200	5.6	291.0	Negative
AB-1	68.0 – 72.0	1200	--	315.0	Negative
AB-4	33.0 – 35.0	900	5.5	132.3	Negative
RW-3	16.0 – 20.0	4,900	3.9	303.0	Negative
RW-7	12.0 – 16.0	650	5.8	246.0	Negative

Laboratory testing for the bulk bag sample consisted of Moisture Density Relationship Test and California Bearing Ratio (CBR) test. Results of the bulk bag sample testing are summarized in Table 3.8. Detailed test results including the moisture density curve and CBR results are included in Appendix C

Table 3.8 – Summary of Moisture Density Relationship Testing

Boring No./ Sample No.	Depth (ft)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Natural Moisture Content (%)	CBR
R-7 / Bulk	2.0 – 5.0	118.7	10.4	4.0	7.9
R-8 / Bulk	0.0 – 5.0	125.5	9.9	17.2	4.4
R-9 / Bulk	1.0 – 10.0	96.7	16.5	11.0	9.5
R-10 / Bulk	1.0 – 5.0	121.4	11.4	10.2	4.4
R-11 / Bulk	1.0 – 5.0	99.2	18.1	22.1	9.5

CBR: California Bearing Ratio at 95% Maximum Dry Density

The unconfined compressive strength and the stress-strain relationship of rock core samples was determined in the Laboratory. The Uniaxial Compressive Strength of rock core samples are summarized in Table 3.9. The stress-strain curves for the rock core tests are included in Appendix C.

Table 3.9 – Summary of Uniaxial Compression Tests

Boring No.	Run No.	Depth (ft)	Rock Type	Compressive Strength, PSI	Young's Modulus @ 50% of Ultimate Strength (Secant Modulus), PSI
AA-1	R-2	129.0	GNEISS	14,309	2.8E6
AA-1	R-3	135.0	GNEISS	29,890	3.7E6
AB-2	R-1	128.0	GNEISS	28,068	1.9E6
AB-2	R-2	133.0	GNEISS	30,362	1.8E6
AB-3	R-1	102.0	GNEISS	25,519	3.0E6
AB-3	R-1	107.5	GNEISS	12,912	1.1E6
AB-4	R-1	108.8	GNEISS	11,106	1.1E6
AB-4	R-2	114.5	GNEISS	18,518	2.5E6
SA-1A	R-1	126.0	GNEISS	13,402	2.3E6
SA-1A	R-2	131.4	GNEISS	20,399	2.7E6
P1-1	R-7	112.0	GNEISS	16,672	3.0E6
P1-2A	R-7	109.0	GNEISS	9,783	1.6E6
P2-1	R-1	89.4	GNEISS	16,416	2.6E6
P2-2	R-6	112.0	GNEISS	5,108	8.8E5

4 SUBSURFACE CONDITIONS

4.1 GEOLOGY

According to the *Geology of Wilmington Area, Delaware, Geologic Map Series No. 4,* the project site is located in the Atlantic Coastal Plain Physiographic Province. According to the Geologic Map of New Castle County, Delaware, Geologic Map Series No. 13 (Kelvin W. Ramsey, 2005) the surficial soils located within the western half of the project site is mapped as the Delaware Bay Group and the eastern half is mapped as the Scotts Corners Formation of the Upper Pleistocene Epoch underlain by the Potomac Formation.

Delaware Bay Group soils are characterized as grayish brown silt overlying a fine to medium silty quartz sand. Natural soils in the region consists of Pleistocene alluvial, swamp, marsh and estuarine deposits along the Christina River; lithologies from organic-rich silty clay and peat to sandy gravel.

Scotts Corners Formation consists of heterogeneous unit of light gray to brown to light-yellowish-brown, coarse to fine sand, gravelly sand and pebble gravel with rare discontinuous beds of organic-rich clayey silt, clayey silt, and pebble gravel.

The Potomac Formation sediments in northern Delaware are believed to have been deposited in a vast alluvial plain by a network of rivers during the Cretaceous. The formation is primarily composed of fine-grained materials in over-bank interfluvial facies, with laterally discontinuous fluvial sand forming a three-dimensional labyrinth in the flood plain muds.

The Potomac Formation has been subjected to high levels of preconsolidation imparted by the weight of younger deposits that have since been eroded away. Characterizing the physical properties of the formation is complicated by the interfluvial mode of deposition, the erratic presence of discontinuous channel and overbank sands, and degradation of the silt and clay properties by weathering processes, which could extend to variable depths.

These Coastal Plain sediments overlay residual soil and bedrock. Bedrock located near the project site is mapped as the Wilmington Complex which may be of Precambrian age. The formation consists of Hypersthene-quartz andesine Gneiss, with minor biotite and magnetite.

Residual soils are soils which have formed in place by the weathering of the parent bedrock. Residual soils typically form a profile characterized by a change from soil to decomposed rock to rock with increasing depths below the ground surface.

4.2 SUBSURFACE CONDITIONS

The Summary of Boring Data and Test Boring Logs, provided in Appendices A and B, respectively, provide details related to the subsurface conditions encountered in the various borings. The stratification lines shown on the Test Boring Logs and Summary of Boring Data represent approximate transitions between material types. In situ, strata changes could occur gradually or at slightly different levels. Also, the borings depict conditions at particular locations and at the particular times indicated. Some conditions, particularly groundwater conditions between borings could vary from the conditions encountered at the particular boring locations.

The contacts between the strata described below generally are not horizontal or well defined. Near rock fractures, the transitions can be very abrupt over a short horizontal distance. Weathering and softer, wetter soil is generally deeper adjacent to fractures, shear zone and lineaments. These discontinuities transmit water much more freely than in the intact rock mass. Weathering will proceed inward from the discontinuities producing deep soft seams alternating with seams of hard weathering rock.

In general, the subsurface materials encountered were broken into five strata as defined below for this report:

- **FILL**
- **Stratum Ia:** Upper Fine Grained Soil
- **Stratum Ib:** Upper Coarse Grained Soil
- **Stratum II:** Potomac Formation – Fine Grained Soil
- **Stratum III:** Residual Soil
- **Stratum IV:** Completely Weathered Rock
- **Stratum V:** Wilmington Complex – Gneiss

FILL: Fill material was encountered in all the land borings except borings R-15, R-16, R-18, and SWM-1. Fill material was also encountered in borings W-1 and W-4 along the west bank of the river. The depth of Fill typically ranged from 2-ft to 16-ft below the existing ground surface. Fill material typically consisted of very loose to dense Sand with varying percentages of Silt and

Clay [USCS: SP, SM, SC, ML, CL]. The natural moisture content ranged from 6 to 106 percent. The stratum consists of mostly non plastic soils. However, the liquid limit for soil samples exhibiting plasticity ranges from 17 to 85, the plastic limit ranges from 18 to 69, and the plasticity index ranges from 2 to 17.The SPT N-values typically ranged from 2 blows per foot (bpf) to 48-bpf and averaged 13-bpf. The corrected SPT N_{60}-values typically ranged from 2-bpf to 68-bpf and averaged 16-bpf.

Table 4.1 summarizes the depth of FILL material encountered in the borings.

Table 4.1 – Summary of FILL Depths

Boring No.	Ground Surface Elevation	Thickness of FILL (ft)	Bottom of FILL Elevation
AA-1	+3.8	6	-2.2
AA-2	+4.3	4	+0.3
AB-1	+6.0	6	0.0
AB-2	+6.0	6	0.0
AB-3	+8.0	6	+2.0
AB-4	+8.2	8	+0.2
AB-5	+6.0	6	0.0
RW-1	11.5	12	-0.5
RW-2	11.3	16	-4.7
RW-3	+6.0	8	-2.0
RW-4	+6.0	8	-2.0
RW-5	+6.2	6.5	-0.3
RW-6	+4.6	6	-1.4
RW-7	+5.3	6	-0.7
RW-8	+4.9	2	+2.9
RW-9	10.9	6	+4.9

Table 4.1 – Summary of FILL Depths

Boring No.	Ground Surface Elevation	Thickness of FILL (ft)	Bottom of FILL Elevation
SA-1	10.8	13	-2.2
SA-2	11.0	10	+1.0
SA-3	10.8	8	+2.8
R-7	+8.0	4	+4.0
R-8	+9.0	6	+3.0
R-9	+5.0	6	-1.0
R-10	+5.0	8	-3.0
R-11	+4.0	6	-2.0
R-12		4	
R-13		2	
R-14		6	
R-17		4	
SWM-2		2	
SWM-3	+5.0	8	-3.0
W-1	-1.5	2	-3.5
W-4	-1.5	2	-3.5

Stratum Ia – Upper Fine Grained Soil: The natural soils at the site generally consisted of soft to medium stiff Highly Plastic Silt and Clay with varying percentages of Sand [USCS: MH, CH, CL, SC-SM] [AASHTO: A-7-5, A-7-6]. The thickness of Stratum Ia ranged from 4-ft to 36-ft. The SPT-N values typically ranged from Weight of Rod (WOR) to 11-bpf and averaged 2-bpf. The corrected SPT N_{60}-values typically ranged from WOR to 11-bpf and averaged 1-bpf. The moisture contents ranged from 12 percent to 162 percent and averaged 73 percent. The liquid limit for the stratum ranges from 21 to 85, the plastic limit ranges from 16 to 69, and the plasticity index ranges from 1 to 38.

The undrained shear strength for this stratum ranges from 29-psf to 701-psf and the effective friction angle ranges from 12.1 degree to 43.9 degrees. The shear strength test results are summarized in Table 3.5. The preconsolidation pressure for the stratum ranges from 0.49-tsf to 2.19-tsf, the Over Consolidation Ratio (OCR) ranges from 0.4 to 2.1, the compression ratio ranges from 0.136 to 0.245, and the recompression ratio ranges from 0.006 to 0.043. The consolidation test results are summarized in Table 3.6.

Stratum Ib – Upper Coarse Grained Soil: This stratum was encountered below Stratum Ia in all the borings and generally consisted of loose to very dense Sand and Gravel with varying percentages of Silt and Clay [USCS: SP, SM, SC, SC-SM] [AASHTO: A-1-a, A-1-b, A-2-4]. The thickness of Stratum Ib ranged from 10-ft to 15-ft. The SPT-N values typically ranged from 7-bpf to 89-bpf and averaged 29-bpf. The corrected SPT N_{60}-values typically ranged from 7-bpf to 88-bpf and averaged 33-bpf. The moisture contents ranged from 8 percent to 47 percent and averaged 17 percent. This stratum consisted of mostly non-plastic soils. However, the liquid limit for a few soil samples exhibiting plasticity ranges from 19 to 48, the plastic limit ranges from 11 to 25, and the plasticity index ranges from 4 to 30.

Stratum II – Potomac Formation Fine Grained Soil: This stratum was encountered below Stratum Ib in all the bridge and retaining wall borings. The Stratum generally consists of stiff to Hard Silt and Clay with varying percentage of Sand [USCS: CL-ML, CL, ML] [AASHTO: A-6, A-7-6, A-4]. The thickness of Stratum Ib ranged from 10-ft to 15-ft

The SPT-N values typically ranged from 11-bpf to 44-bpf and averaged 25-bpf. The corrected SPT N_{60}-values typically ranged from 7-bpf to 53-bpf and averaged 25-bpf. The moisture contents ranged from 12 percent to 40 percent and averaged 27 percent. The liquid limit for the stratum ranges from 22 to 85, the plastic limit ranges from 12 to 50, and the plasticity index ranges from 4 to 54. The preconsolidation pressure for one sample from this stratum was 0.62-tsf, the compression ratio was 0.105 and the recompression ratio was 0.039. The consolidation test result is included in Table 3.6.

Stratum III – Residual Soil: Stratum III was encountered in all the bridge and retaining wall borings below Stratum II. The residual soils at the site generally consisted of loose to dense SAND with varying percentages of Silt and Clay (USCS: SM, SC) [AASHTO: A-2-4, A-2-5, A-2-6, A-2-7] and stiff to hard medium plastic Silt and Clay (USCS: CL, CH, MH) [AASHTO A-5, A-6, A-7-5, A-7-6].

The SPT N-values typically ranged from 19-bpf to 57-bpf with an average SPT N-value of 42-bpf. The corrected SPT N_{60}-values typically ranged from 6-bpf to 82-bpf and averaged 33-bpf. The moisture contents ranged from 18 percent to 47 percent and averaged 29 percent. The liquid limit for the stratum ranges from 24 to 75, the plastic limit ranges from 21 to 47, and the plasticity index ranges from 8 to 34.

Stratum IV – Completely Weathered Rock: This stratum was encountered below Stratum III. This stratum is an Intermediate Geo-Material (IGM) described as Sand and Silt with varying percentage of Gravel-sized rock fragments and Clay. Completely Weathered Rock (CWR) requiring coring techniques to sample was encountered in Borings P1-1, P1-2, P2-1, and P2-2 from EL -95 to EL -103. Thin rock seams were encountered.

The moisture contents for this stratum ranged from 11 percent to 37 percent and averaged 25 percent. The liquid limit for the stratum ranges from 31 to 71, the plastic limit ranges from 22 to 45, and the plasticity index ranges from 11 to 42.

Completely Weathered Rock (CWR) is defined in this report as residual material which retains the relic rock structure of the parent bedrock and exhibits SPT N-values consistently in excess of 60-blows/foot and less than 50-blows/inch or auger refusal; rock cores with recoveries less than 40-percent were also defined as CWR in this report. Auger refusal, thus indicated, may result from hard cemented soil, soft weathered rock, coarse gravel or boulders, thin rock seams, or the upper surface of sound continuous rock and is also dependent of the type of drilling machine used during the exploration. There is a wide range of torque and crowd within the typical types of drilling machines utilized in geotechnical exploration. Refusal encountered with a relatively light duty drill rig may be penetrated with a more powerful machine. Rock coring techniques are required to determine the character and continuity of the materials located below the refusal elevation.

Stratum V – Wilmington Complex GNEISS: This stratum was encountered below Strata IV in all the bridge borings. This stratum is described as gray GNEISS, highly weathered to moderately weathered, extremely to slightly fractured, and weak to medium strong rock. This stratum extended to the bottom of boring. The Uniaxial Compressive strength of the bedrock ranges from 5.1-ksi to 30.3-ksi and the Young's Modulus ranges from 880-ksi to 3,700-ksi. The Uniaxial Compression test results are summarized in Table 3.9. Table 4.2 summarizes the Rock Core Recovery Percentage and Table 4.3 summarizes the Rock Mass Rating (RMR) for the pier borings. The RMR for runs with RQD values greater than zero(0) ranged from 29 to 64 indicating Poor to Good rock mass. RMR is an index used to estimate rock mass behavior based on the following parameters. A description of the RMR is included in Appendix B. It was not possible to determine the qu of samples with RQD values of zero (0) because there was not enough intact sample to perform a PLT or qu test.

- Strength of intact rock samples
- Rock Quality Designation (RQD)
- Spacing of joints
- Condition of Joints
- Groundwater conditions
- Orientation of discontinuities

Table 4.2 – Rock Core Recoveries

Borings	Substructure	Rock Recoveries per Rock Core Run (%)									
		R-1	**R-2**	**R-3**	**R-4**	**R-5**	**R-6**	**R-7**	**R-8**	**R-9**	**R-10**
AA-1	Abutment A	75	95	93	----	----	----	----	----	----	----
SA-1A	Abutment A	80	96	86	----	----	----	----	----	----	----
P1-1	Pier 1	95	57	0	92	65	58	100	100	----	----
P1-2A	Pier 1	100	60	90	67	44	0	92	100	97	98
P2-1	Pier 2	89	60	35	80	93	95	97	95	----	----
P2-2	Pier 2	3	0	90	38	100	87	60	80	7	----
AB-2	Abutment B	62	95	98	----	----	----	----	----	----	----
AB-4	Abutment B	74	100	----	----	----	----	----	----	----	----
----- Not Cored		REC <40% is considered CWR									

Table 4.3 – Rock Mass Rating for Pier Borings							
Borings	**Substructure**	**R-5**	**R-6**	**R-7**	**R-8**	**R-9**	**R-10**
P1-1	Pier 1	***	***	64 good	64 good	----	----
P1-2A	Pier 1	***	***	***	59 fair	42 fair	59 fair
P2-1	Pier 2	42 fair	42 fair	57 fair	42 fair	----	----
P2-2	Pier 2	***	34 poor	29 poor	34 poor	***	----
*** RMR calculated only for Runs with RQD values ---- Not Cored							

4.3 GROUNDWATER

Groundwater was encountered at depths ranging from 6 to 8-ft below the existing ground surface. The tidal fluctuation in Christina River is approximately 6.5-ft. A more accurate determination of the hydrostatic water table would require the installation of perforated pipes or piezometers which can be monitored over an extended period of time. The actual level of the hydrostatic water table and the amount and level of perched water should be anticipated to fluctuate throughout the year, depending upon variations in precipitation, surface run-off, infiltration, tidal fluctuation, site topography, and drainage. The tidal fluctuation will have some influence on the water table along the shore line.

It was necessary to backfill the borings immediately after the completion of drilling to provide safe conditions because the borings were located in areas frequented by pedestrians. In cases where the boring was immediately backfilled with a tremie grout, the boring logs note the depth where groundwater was observed either within the recovered soil sample, on the split barrel samples, on the drill rods, or in the soil brought to the surface by the hollow stem augers.

5 EVALUATIONS AND RECOMMENDATIONS

The following recommendations have been developed on the basis of the previously described project characteristics and subsurface conditions.

5.1 BRIDGE FOUNDATION ALTERNATIVE ANALYSIS

The primary purpose of the bridge foundation alternative analysis was to assess suitable foundation types relative to the applied loads, physical constraints of the site, and the subsurface conditions that were encountered during the subsurface exploration.

The following bridge foundations were evaluated for construction.

- Steel H-Piles
- Steel Pipe Piles
- Drilled Shafts
- Prestressed-Precast Concrete (PPC) Piles

5.1.1 Steel H-Piles

Low-displacement steel H-piles provide the advantage of flexibility for site conditions where end bearing is likely to vary abruptly. H-piles can be easily spliced, they are available in various sizes, and they provide high capacity with minimal displacement, noise, and vibration compared to displacement piles. The use of driven pile foundation eliminates spoil generation and disposal.

The disadvantages to steel H-piles are they may be damaged or deflected by obstructions and the capacity of individual piles is smaller than that of pipe piles and drilled shafts. H-piles have smaller section modulus and will require battered piles for lateral design. Steel piles are subject to corrosion.

Stratum Ia below the fill layer extends to depths ranging from approximately 10 to 40-ft below the ground surface and will provide negligible lateral support to the pile foundation. Unsupported pile lengths of as much as 40-ft extending through Stratum Ia is a concern for slender piles and may not satisfy the slenderness ratio requirement. The construction of the ramps on either side of the bridge may cause lateral spreading of the soft soil if normal weight materials are used, and H-piles may not be able to resist the bending forces induced by the lateral spread.

5.1.2 Steel Pipe Piles

The advantages of a steel pipe pile are the availability of a wide selection of diameters and wall thicknesses. The length of the pipe pile can be easily extended by splicing as necessary in response to variable subsurface conditions. The use of a driven pile foundation eliminates spoil generation and disposal. Open ended pipe piles provide high capacity with minimal displacement, thereby reducing the risk of dislocating nearby piles. Pipe piles possess a higher (and directionally uniform) moment of inertia beneficial to the resistance to bending.

Driving may be difficult in hard or very dense soils. Boulders and ledges of rock could damage the pile; however, if needed, the soil plug could be drilled out and the pipe pile inspected with a drop light or camera. There is a high degree of uncertainty of plug formation and plug response under static and dynamic loading in open ended pipe piles. Steel piles are subject to corrosion.

Table 5.1 details the depth of pile installation and estimated bottom of pile elevations at the abutment and piers. It should be noted that piles within Abutment A will be battered at 3H:12V for support of the lateral loads from the abutment.

Table 5.1 – Pipe Pile Tip Elevations (24-inch)

Location	Length from BOF (ft)	Estimated Pile Tip Elevation
Abutment A	81	-78
Pier 1	73	-99
Pier 2	85	-111
Abutment B	75	-75

5.1.3 Drilled Shafts

The advantages of a drilled shaft foundation are a single shaft can replace a group of piles due to the high axial and lateral loading capacities. Drilled shafts are easily adaptable to varying site conditions during installation and generate less noise and reduced vibrations compared to driven pipe piles. The bottom of a shaft can be visually inspected during construction with a televiewer or a down-hole inspector.

The disadvantages of a drilled shaft foundation included the mobilization of larger equipment. Drilled shafts are extremely sensitive to construction procedures and they generate large quantities of spoils and will incur additional cost for hauling and disposal. Some of the subsurface materials are contaminated and hauling these materials will significantly increase cost and potentially require special drilling procedures to avoid environmental damage or injury to field crews. Rock seams, boulders, and ledges may cause difficulty in drilled shaft construction. Drilling through boulders and broken rock could add significant construction time and delay to the overall project. Static load testing of drilled shafts using Osterberg Cell Tests (OCT) or Statnamic Load Tests (SNLT) add a significant cost increase. Dewatering with full length casing will be required unless drilling slurry is used. If a slurry is used the skin friction could be reduced.

5.1.4 Prestressed-Precast Concrete (PPC) Piles

The advantage of Prestressed-Precast Concrete (PPC) Piles is they typically are less expensive per unit ton of load carrying capability than most other piles. Typically they can be driven to shallower depths than other driven piles, due to the closed end of the piles the area of contact for end bearing is consistent and thus less conservative calculations can be made for the end bearing resistance of the piles compared to pipe piles. PPC provide the same moment of inertia and thus provide increased lateral resistance than other driven pile foundations. The use of driven piles will eliminate spoil generation and disposal. The estimated tip elevation of the piles is at a higher elevation that for pipe piles reducing driving time and reducing the risk of hitting obstructions in Stratum IV.

The disadvantage of PPC's include the inability to effectively field splice the piles if the piles are required to be driven to a deeper depth, in stiff overburden materials there may be the need to pre-drill. PPC piles are heavier than steel pipe piles or H piles, and therefore necessitate the use of larger cranes and possibly larger driving equipment. PPC are more prone to damage during driving than other piles, thus steps should be taken to minimize this potential.

Preliminary analysis indicates that 24-inch PPC piles at the pier locations and 18-inch PPC piles at the abutments would need to be driven to a depth of approximately 60 and 90-ft respectively below the proposed bottom of footing.

5.2 BRIDGE APPROACH RETAINING WALL FOUNDATION ALTERNATIVE ANALYSIS

The primary purpose of the bridge approach retaining wall foundation alternative analysis was to assess suitable foundation types relative to the physical constraints of the site and the subsurface conditions that have been encountered during the subsurface exploration.

The following alternatives were evaluated for construction of the bridge approaches at the Christina River Bridge.

1. Conventional Abutment with Extended CIP Wingwalls
2. U-Shaped Cast-In-Place (CIP) Concrete Wall Supported on Drilled Shafts
3. Preloading Embankment with Mechanically Stabilized Earth (MSE) Walls
4. EPS Approaches with Conventional CIP Abutment and Wingwalls Supported on Drilled Shafts
5. Deep Mixing Method (DMM) with Mechanically Stabilized Earth (MSE) Walls
6. Back Span
7. Stone Columns/Densified Aggregate Piers
8. Tied Sheet Pile Walls
9. Tangent Walls

Due to the presence of normally consolidated, highly plastic silts at the proposed abutment and ramp retaining walls, we anticipate large settlements in excess of 2-ft on the west approach and 3-ft on the east approach using conventional Mechanically Stabilized Earth (MSE) wall abutments and retaining walls. Due to the weak foundation soils the slope stability analysis for the conventional MSE walls also indicated that the Factor of Safety (FS) was less than the project criteria. We evaluated the alternatives below to recommend a safe and economical design for the approach embankments. The west approach and the east approach will be addressed separately.

5.2.1 West Approach

5.2.1.1 Conventional Abutment with Extended CIP Wingwalls

The abutment wingwalls will be drilled shaft supported and extend the entire length of the approach ramp. Backfilling with common borrow will cause settlements of up to 2-ft; settlements larger than about 0.4-inches will induce downdrag and lateral spreading loads on the drilled shaft foundations.

Advantages

- Drilled shaft supported wingwalls will not cause significant settlement due to the retaining walls
- Scour resistant
- Extended wingwalls will increase global stability

- Utilizes conventional design and construction techniques
- Predictable outcome

Disadvantages

- Backfilling the wingwalls with common borrow will cause settlement and downdrag on the drilled shafts. The drilled shafts will have to be designed for the additional load to reduce differential settlement, or EPS will be required as backfill as described in Option 4 or the embankment between the wingwalls will need to be supported by a drilled shaft supported slab/raft as described in Option 2.
- Excavation for the foundation cap will likely require dewatering and decontamination/disposal of contaminated groundwater and soil
- Additional cost for more drilled shafts and CIP than other options mentioned below
- Time consuming to install a large number of drilled shafts, place forms, erect re-bar, and wait for concrete to cure
- If EPS will be used, it will be lower cost to simply apply permanent facing to EPS and avoid the drilled shafts and CIP for wingwalls.

5.2.1.2 U-Shaped Cast-In-Place (CIP) Concrete Wall Supported on Drilled Shafts

This approach will consist of a drilled shaft supported mat foundation that will support the abutment, wing walls, and backfill.

Advantages

- The U-shaped wall will consist of a drilled shaft supported mat to transfer the embankment loads to deeper, firmer bearing layer to eliminate downdrag and lateral loads on the wall and abutment drilled shafts as well as nearby structures.
- Global stability will not be a concern.
- Scour protection will be minimal as the drilled shafts can be designed for the scour depth.
- The approach width is 45-ft, so the width of the U-wall is not unreasonable.

Disadvantages

- Higher cost
- Longer construction time

5.2.1.3 Preloading Embankment with Mechanically Stabilized Earth (MSE) Walls

Preloading the embankment area with a temporary embankment will increase the shear strength of the foundation soils and will significantly reduce settlements of the permanent embankment by consolidating the soils prior to final construction. The construction of the approach embankment using conventional MSE Wall with #57 stone in the reinforcement zone without preloading will result in settlement in excess of 1.5 to 2-ft as summarized in Table 5.2.

Table 5.2 – Summary of Total Settlements using #57 Stone without Preloading

Location	Centerline	Facing Panel
Abutment A (STA 436+00)	19-inches	19-inches
STA 435+75	25-inches	19-inches

Due to the weak foundation soil the preloading will have to be constructed in multiple stages with quarantine period between stages for the soil to consolidate and gain strength for the next stage of construction. Prefabricated Vertical Drains (PVD's) should be used to accelerate the time rate of settlement to minimize the quarantine period between the embankment construction stages.

The construction of the preloading embankment within 30-ft of the proposed location of the abutment is not feasible as the slope stability analysis indicates that even with thin lifts, multiple stage construction the embankment will not meet the design criteria of a Factor of Safety (FS) greater than 1.2, the design criteria for a temporary embankment. The preloading embankment has to be offset approximately 30-ft from the proposed abutment location. The first 30-ft from the abutment face will only be partially loaded with a 2(H):1(V) embankment slope, so another treatment method will be required within 30-ft of the abutment. The construction of the first 30-ft of approach behind the abutment may include conventional drilled shafts supported CIP wingwalls, but will also need to be backfilled with Expanded Poly-Styrene (EPS) to avoid downdrag, lateral spreading loads on the drilled shafts, and global stability difficulties. A back-span could also be used. See Option 5.

The preloading embankment will have to be constructed in 4-stages with the maximum height of 15-ft at approximately STA 435+70. The quarantine period between each preloading stage will be approximately 60-days each. We estimate approximately 10-months for the preloading of the foundation soils. CPT's should be performed between stages to confirm the strength gain has been enough to allow for construction of the next stage. The staging program can be modified, either accelerated or slowed, based on the instrumentation and CPT probe results.

After the desired consolidation and strength gain has occurred and the settlement has leveled off the temporary surcharge embankment will be removed, drilled shafts installed for the abutment and wing walls and the MSE wall constructed.

Secondary consolidation or creep of fine grained soil will occur due to the rearrangement of the soil particles resulting in the change in volume of the soil mass. After the pore pressure has dissipated and completion of primary consolidation secondary consolidation will follow. The rate of secondary consolidation is very slow and will continue to occur during the design life of the permanent structure. Secondary consolidation settlement will be relatively uniform as it is independent of the stress level. We estimate approximately 3 to 4-inches of secondary consolidation spanning the design life of the structure. The effects of the secondary consolidation settlement can be minimized by using shorter parapet segments, and frequent vertical slip joints in the wall facing panels.

A comprehensive instrumentation plan will be required during the construction of the temporary embankment to monitor the settlement of the foundation soil and surrounding structures. The instrumentation plan will consist of settlement monitoring plates along the preloading embankment, settlement monitoring points on existing structures in close proximity to the temporary embankment and the installation of piezometers to monitor the pore-water pressure developed within the foundation soil during the construction of the embankment. Monitoring of the pore-water pressure will help in determining the maximum height of embankment fill that can be constructed in each stage of construction.

Advantages

- The preloading and PVD's can be built using an early contract so that the site can be fully preloaded prior to NTP for the bridge contract.
- Ease of construction
- Cost savings over using a U-wall
- Only minor excavation required, so little risk of excavating contaminated soil
- Utilizes conventional techniques

Disadvantages

- Cost of an advance contract
- Preloading will have to be done in several stages due to global stability concerns

- Difficult to accurately predict time rate of settlement and shear strength gain due to surcharging making the length of the quarantine period difficult to estimate
- If the quarantine period needs to be extended it could interfere the schedule and cost for the main contract
- Quarantine period will last around a year
- Installation of PVD's could cause smearing of the soils increasing the quarantine period
- Significant instrumentation required
- Requires hauling in and hauling off or stockpiling a significant amount of fill for the temporary embankment and then hauling in more soil and materials for the permanent embankment and MSE walls

5.2.1.4 Load Balancing (EPS Embankment) with Conventional Abutment and Wingwalls supported on Drilled Shafts

Load balancing would be achieved by excavating the in situ soil and disposing of it off site or re-using it on site and then constructing the embankment out of Expanded Poly-Styrene (EPS). Complete load balancing will require 4.5-ft of excavation of the in situ soil to approximately EL +6.0 and building the embankment with EPS with a precast or a tilt-up concrete fascia panels. Abutments placed on EPS fills cannot support lateral loads from typical deep foundation for stub abutments; therefore, the bridge abutment will have to consist of a full height conventional abutment founded on a deep foundation. The first 26-ft of wingwalls, to the pedestrian underpass, behind the abutment should be constructed of CIP concrete wall supported on a drilled shaft foundation with the underpass from STA 435+45 to 435+74 and the rest of the approach embankment wall constructed of EPS Retaining walls. The approach beyond STA 434+00 will consist of an embankment constructed of common borrow. The drilled shaft supported wingwalls and the use of complete load balancing with EPS adjacent to the abutment will not induce any additional activating force along the slope perpendicular to the river and minimize down drag on the drilled shafts.

EPS backfill will be required between the CIP wingwalls to minimize settlement of the foundation soils that may induce downdrag and lateral spreading loads on the drilled shafts foundation. The EPS MSE wall adjacent to the CIP wingwall may cause some downdrag on the drilled shafts if the load is not completely balanced with EPS and undercutting, but the downdrag will be significantly less than if common borrow were to be used as backfill.

Assuming 3-ft of undercutting and embankment height of 9-ft the embankment weight will not be fully compensated. We estimate the settlement at the abutment will be as shown in Table 5.3.

Table 5.3 – Summary of Total Settlements using EPS (3-ft of Undercutting)		
Location	**Centerline**	**Facing Panel**
Abutment A (STA 436+00)	1.7-inches	1.1-inches
STA 435+75	3.2-inches	1.2-inches

It is expected to take 2-years to reach a point with less than 1-inch of settlement remaining at the centerline of the embankment at STA 435+75. PVDs may be used to accelerate the settlement process. Alternatives to using PVD's would be to delay installation of the parapets and final paving, using shorter parapet segments, and frequent vertical slip joints in the wall facing panels. These settlements will cause downdrag on the drilled shafts. This can be accommodated in the design of the foundation by longer shafts. Settlements should be monitored to verify the predictions.

To be fully compensating the undercutting will need to extend 4.5-ft below the existing ground surface. A complete load balance can be achieved using EPS along the west approach ramp for the segment west of the underpass. A complete load balance for the approximately 26-ft segment between the underpass and the abutment will require excavating a significant depth below the river elevation. The undercutting will extend to approximately EL +1 at the abutment location where the ditch is located. This will extend the excavation below the river elevation during high tides and will require a cofferdam for the excavation and construction of the abutment foundation.

Advantages

- Settlement after construction will be significantly less than other options. Generally, EPS can handle 1 to 2-ft of post construction settlement provided the settlement is relatively uniform along the ramp.
- Reasonable cost
- Fast construction
- Minimizes down-drag loads on abutment drilled shafts
- Maintains slope stability at the abutment

Disadvantages

- Will require excavating to below the groundwater table, stockpiling or transporting/treating/disposal of contaminated soils, and dewatering/treating potentially contaminated water. The disposal quantity can be minimized if the excavated soil can be re-used onsite, but this will require room for stockpiling.

- May still cause some down-drag on drilled shaft supported abutment and wingwalls
- Utilizes unusual construction material with unusual design and construction techniques

5.2.1.5 Deep Mixing Method with Mechanically Stabilized Earth (MSE) Walls

Deep Mixing Method (DMM) is a ground improvement techniques to treat the in situ soil with cementitious or other binders to enhance the engineering properties of the native soil. The treated soils using DMM have increased strength and reduced compressibility. The DMM columns have higher stiffness and transfer the embankment load through the soft soils into the underlying competent foundation soils.

The DMM design was based on 4-ft diameter soil mix columns with a shear wall arrangement perpendicular to the face of the ramp walls and isolated columns in the interior of the approach embankment. This arrangement is shown in Figure A-5. The soil mix columns extend through the soft soil, Stratum Ia, into the medium dense to dense soil, Stratum Ib, underlying Stratum Ia. Table 5.4 summarizes the details of the DMM arrangement for the west approach. The 28 day UCC strength typically ranges for 100 to 300-psi and depends on the contractor's means and methods of mixing as well as the reaction of the cement used with the in situ soils.

<table>
<tr><th colspan="8">Table 5.4 –Summary of Deep Mixing Method Parameters</th></tr>
<tr><td colspan="4">28 day (UCC)</td><td colspan="4">120-psi</td></tr>
<tr><td colspan="4">Soil Mix Column Diameter</td><td colspan="4">4-ft</td></tr>
<tr><td colspan="4">Shear Wall Overlap Ratio</td><td colspan="4">0.3 (1.2-ft overlap for 4-ft diameter columns)</td></tr>
<tr><th rowspan="2">Location</th><th rowspan="2">Start</th><th rowspan="2">End</th><th colspan="3">Shear Wall</th><th colspan="2">Isolated Columns</th></tr>
<tr><th>Length (ft)</th><th>C-C Spacing</th><th>Area Ratio</th><th>C-C Spacing</th><th>Area Ratio</th></tr>
<tr><td rowspan="4">West Approach Ramp</td><td>STA 435+75</td><td>STA 436+00</td><td colspan="5">Deep Mix Columns in rectangular grid or lattice pattern</td></tr>
<tr><td>STA 435+40</td><td>STA 435+75</td><td colspan="5">Underpass supported on Drilled Shaft</td></tr>
<tr><td>STA 435+00</td><td>STA 435+40</td><td>18.0-ft</td><td>10.0-ft</td><td>0.36</td><td>7.0-ft</td><td>0.25</td></tr>
<tr><td>STA 434+00</td><td>STA 435+00</td><td>18.0-ft</td><td>12.0-ft</td><td>0.30</td><td>7.0-ft</td><td>0.25</td></tr>
</table>

Advantages

- Soil improvement will transfer the load from the embankment to stiffer and denser foundation soil below the soft soils as well as improve the soft foundation soils
- Soil improvement will limit the settlement of the foundation soil
- Soil improvement will limit downdrag and lateral spreading on the foundation drilled shafts for the bridge abutment
- The approaches can consist of conventional MSE wall with moment slab and barriers on top. No need for unusual design elements such as flexible connection for the EPS approaches
- Reduced depth of excavation below existing ground surface for the construction of the bridging layer (3-ft). Least amount of undercutting.
- Spoils from soil mixing will be cement treated soils and can be used as embankment fill

Disadvantages

- Higher Cost
- Area required to stock pile soil mix spoils
- Requires experienced specialty contractor to prepare final design and to construct
- A bench scale testing program is required during design phase to provide the bidders with adequate information to prepare a bid

5.2.1.6 Back Span

An additional bridge span may be added to move the bridge approach further away from the river. The additional span will also reduce the maximum height of the approach ramp retaining walls. DelDOT requires a 4-ft clearance below the superstructure for inspection purposes. A 50-ft back span can be constructed from the original abutment location. The approach ramps may be constructed using one of the five options listed above.

Advantages

- The reduced height of the approach ramp may reduce one preloading stage or a significant volume of EPS.
- The additional distance from the river will help in the dewatering process during construction.

Disadvantages

- Extra cost for the construction of the additional span.
- Cost for the inspection and maintenance of the additional span throughout the life of the bridge.

- Aesthetically, the spans will not be balanced unless the back span is built with curtain walls

5.2.1.7 Stone Columns/Densified Aggregate Piers

Stone columns were evaluated as a means of supporting the approach MSE walls. However, treatment of in situ soils with stone columns will not significantly reduce settlement or improve stability enough to comply with design criteria without a large replacement ratio. Even with a replacement ratio of 26-percent the resulting settlement is in excess of 12-inches. Replacement ratios of more than 30 to 35-per cent usually are not cost effective. Excavation for columns will require large amounts of potentially contaminated soil and water to be treated/transported/disposed. To construct densified aggregate piers will require keeping the drilled excavation open for the full depth in order to place and compact the stone backfill. The soft and wet soils will not likely stay open long enough unless bottom-feed stone columns are used.

5.2.1.8 Tied Sheet Pile Walls and Tangent Pile Walls

Tied sheet pile walls and tangent piles walls were evaluated to avoid affecting the existing mall building prior to the relocation of the roadway. Since the roadway has been relocated further from the existing building, these options do not contribute to reducing the settlements or improving the stability of the foundations; therefore, these options were not developed further.

5.2.2 East Approach

5.2.2.1 Conventional Abutment with Extended CIP Wingwalls

The abutment wingwalls will be supported on drilled shafts and extend the entire length of the ramp walls. Backfilling with common borrow will cause settlement of up to 3-ft; settlements larger than about 0.4-inches will induce downdrag and lateral spreading loads of the drilled shaft foundations.

Advantages

- Drilled shaft supported wingwalls will not cause significant settlement due to the retaining walls
- Extended wingwalls will maintain increase global stability
- Scour resistant
- Utilizes conventional design and construction techniques
- Predictable outcome

Disadvantages

- Backfilling the wingwalls with common borrow will cause settlement and downdrag on the piles. The drilled shafts will have to be designed for the additional load to reduce differential settlement, or EPS will be required as backfill as described in Option 4 or the embankment between the wing walls will need to be supported by a drilled shaft supported slab/raft as described in Option 2.
- Excavation for foundation cap will likely require dewatering and decontamination/disposal of contaminated groundwater and soil
- Additional cost for more drilled shafts and CIP
- Time consuming to drive a large number of piles, place forms, erect re-bar, and wait for concrete to cure
- If EPS will be used, it will be lower cost to simply apply permanent facing to EPS and avoid the drilled shafts and CIP for the extended wingwalls.

5.2.2.2 U-Shaped Cast-In-Place (CIP) Concrete Wall Supported on Drilled Shaft

A drilled shaft supported U-wall on the East approach is not recommended because the east approach embankment is significantly taller and wider than on the west approach and will require the wall to extend a significant distance from the abutment. The ramp walls extend approximately 230-ft from the face of the abutment. The east approach embankment is approximately 45-ft wide at the abutment face and flares up to approximately 55-ft at STA 443+00. This option will therefore be significantly more expensive.

5.2.2.3 Preloading Embankment with Mechanically Stabilized Earth (MSE) Walls

Preloading the embankment area with a temporary embankment will increase the shear strength of the foundation soils and will significantly reduce settlements of the permanent embankment by consolidating the soils prior to the final construction. The construction of the approach embankment using conventional MSE Wall with #57 stone backfill without preloading will result in settlement in excess of 3-ft as summarized in Table 5.5.

Table 5.5 – Summary of Total Settlements using #57 stone without Preloading		
Location	**Centerline**	**Facing Panel**
Abutment B (STA 440+70)	29-inches	17-inches
STA 441+20	41-inches	27-inches

Due to very weak foundation soil the preloading embankment will have to be constructed in multiple stages with quarantine period in between stages for the soil to consolidate and gain strength for the next stage of construction. Prefabricated Vertical Drains (PVD's) should be used to accelerate the time rate of settlement to minimize the quarantine period between the embankment construction stages.

The construction of the preloading embankment within 45-ft of the proposed location of the abutment is not feasible as the slope stability analysis indicates that even with thin lifts, multiple stage construction the embankment will not meet the design criteria of a Factor of Safety (FS) greater than 1.2 for a temporary embankment. The preloading embankment has to be offset approximately 45-ft from the proposed abutment location. The first 45-ft from the abutment face will only be partially loaded with a 2(H):1(V) embankment slope, so another treatment method will be required within 45-ft of the abutment. The construction of the first 45-ft of approach behind the abutment may include a conventional abutment and wingwall supported on drilled shafts, but will also need to be backfilled with Expanded Poly-Styrene (EPS) to avoid downdrag, lateral spreading loads on the drilled shafts, and global stability difficulties.

The preloading embankment will have to be constructed in six stages with the maximum height of 21-ft at approximately STA 441+15. The quarantine period between each preloading stages will be approximately 60-days. We estimate approximately 15-months for the preloading of the foundation soils. CPT's should be performed between stages to confirm the strength gain has been enough to allow for construction of the next stage. The staging program can be modified, either accelerated or slowed, based on the instrumentation and CPT probe results.

After the desired consolidation and strength gain has occurred and the settlement has leveled off the temporary surcharge embankment will be removed, drilled shafts installed for the abutment and wingwalls and the MSE wall constructed.

We estimate approximately 4 to 5-inches of secondary consolidation settlement spanning the design life of the structure. The effects of the secondary consolidation settlement can be minimized by using shorter parapet segments, and frequent vertical slip joints in the wall facing panels.

A comprehensive instrumentation plan as described for the west approach will be required during the construction of the temporary embankment.

Advantages

- The preloading and PVD's can be built using an early contract so that the site can be fully preloaded prior to NTP for the bridge contract.
- Settlement of permanent walls within acceptable limits.
- Ease of construction.
- Cost savings over using a U-wall.
- Only minor excavation required so little risk of excavating contaminated soil.
- Utilized conventional techniques.

Disadvantages

- Cost of an advance contract.
- Preloading will have to be done in stages due to global stability concerns.
- Difficult to accurately predict time rate of settlement and shear strength gain due to surcharging making the length of the quarantine period difficult to estimate.
- If the quarantine period needs to be extended it could interfere with the schedule and cost for the main contract.
- Quarantine period will last around 15-months.
- Installation of PVD's could cause smearing of the soils increasing the quarantine period.
- Significant instrumentation required.
- Requires hauling in and hauling off or stockpiling a significant amount of fill for the temporary embankment and then hauling in more soil and materials for the permanent embankment and MSE walls

5.2.2.4 EPS Approach with Conventional Abutment and Wingwalls Supported on Drilled Shafts

The early contract for the construction of the preloading embankment can be avoided by the construction of the entire ramp using EPS with a precast concrete fascia. EPS is generally 1/100th the weight of normal backfill soil. A partial load balancing will be required to achieve the required Factor of Safety (FS) for bearing capacity of the ramp fill. The deepest the existing soils can be undercut without encountering ground water is 2.5-ft. As described below, this will not be adequate to completely compensate for the weight of the new fill; therefore excavations will need to extend below groundwater.

Abutments placed on EPS fills cannot support lateral loads from a typical deep foundation for stub abutments; therefore, the bridge abutment will have to consist of a full height conventional abutment founded on a drilled shaft foundation. The first 30-ft of wingwalls, from STA 440+70 to 441+00, behind the abutment will be constructed of CIP concrete wall supported on a drilled shaft foundation with the rest of the approach embankment wall from STA 441+00 to STA 443+00 constructed of EPS Retaining walls. The approach beyond STA 443+00 will consist of an embankment constructed of EPS to STA 444+00 and with common borrow beyond that. The 30-ft of drilled shaft supported wingwalls and the use of load balancing using EPS adjacent to the abutment will reduce the activating force along the slope perpendicular to the river.

EPS backfill will be required between the CIP wingwalls to minimize settlement of the foundation soils that may induce downdrag and lateral spreading loads on the drilled shaft foundation. The EPS retaining wall adjacent to the CIP wingwall may cause some downdrag on the drilled shafts if complete load balancing is not achieved, but the downdrag will be significantly less than if common borrow were to be used as backfill.

Achieving a total load balance is not feasible at the east approach due to the 100-year flood elevation. The EPS embankment is designed for a partial load balance with the undercut of the foundation soil extending to EL -0.5. Undercutting deeper and backfilling with EPS will cause a larger buoyancy force requiring more overburden on top. The design of the EPS embankment to resist the uplift forces during a 100 year flood will require approximately 680-psf of overburden above the EPS. The resulting additional surcharge on the foundation soil is 240-psf. Table 5.6 summarizes the settlement of the foundation soils due to the weight required to provide adequate buoyancy resistance.

Table 5.6 – Summary of Settlement EPS East Approach			
Surcharge Load	**Elastic Settlement**	**Primary Consolidation Settlement**	**Secondary Consolidation Settlement**
240-psf	0.5-inches	9-inches	4.5-inches

The settlement will induce downdrag on the abutment and wing wall drilled shaft foundation and will add lateral loads to the drilled shafts. Preloading the approach ramp area with a temporary embankment will increase the shear strength of the foundation soils and will significantly reduce settlements of the permanent embankment by consolidating the soils prior to the final construction. Preloading will minimize the downdrag and lateral spreading on the foundation. A temporary embankment at least 4-ft high will have to be constructed with a quarantine period for the soil to gain strength and consolidate. Settlements should be monitored with settlement plates and in situ pore pressures should be monitored using vibrating wire piezometers. CPT's should be performed to confirm the strength gain has been achieved before removing the temporary surcharge.

The calculated time of completion of 90-percent of the primary consolidation for the temporary preloading embankment is approximately 1080-days. Prefabricated Vertical Drains (PVDs) should be used to accelerate the time rate of settlement to minimize the quarantine period. The construction of the preloading embankment should extend to the face of the proposed abutment and will require a sheet pile coffer dam and a temporary wire or wrap around face MSE wall or geo-tubes. The limits of the preloading embankment should be from the face of Abutment B to approximately STA 444+00. Table 5.7 summarizes the time rate of settlement of the embankment using PVDs. The quarantine period can be modified, either accelerated or slowed, based on the instrumentation and CPT probe results.

Table 5.7 – Summary of Time Rate of Settlement			
Location	**Time for Completion of 90% Primary Consolidation Settlement**		
	Without PVDs	**PVDs at 10-ft c-c**	**PVDs at 5-ft c-c**
East Approach	1080-days	330-day	95-days

The preloading embankment will require approximately 1100 PVDs installed at a 5-ft center to center grid to a depth of approximately 50-ft below the existing ground surface. The time required to install the PVDs will be approximately 1.5 months and the time required to construct the temporary surcharge will be approximately 1 month. Table 5.8 summarizes the total time required for the temporary preloading.

Table 5.8 – Summary of Construction Time for Temporary Preloading

Description	Installation Time
Install Prefabricated Vertical Drains	1.5 months
Construction of Temporary Embankment	1.0 months
Quarantine Period	3.5 months
Removal of Temporary Embankment	1.0 month
Total	7.0 months

Due to time required to construct the temporary preloading embankment with the PVDs and the length of the quarantine period this option may require an advanced contract, or a notice in the contract that sufficient time should be built into the schedule in case there are delays in removing the temporary surcharge. After the desired consolidation and strength gain has occurred and the settlement has leveled off the temporary surcharge embankment will be removed, drilled shafts installed for the abutment and wing walls and the EPS embankment constructed.

Secondary consolidation or creep of fine grained soil will occur due to the rearrangement of the soil particles resulting in the change in volume of the soil mass. After the pore pressure has dissipated and completion of primary consolidation, secondary consolidation will follow. The rate of secondary consolidation is very slow and will continue to occur during the life of the permanent structure. Secondary consolidation settlement will be relatively uniform as it is independent of the stress level. We estimate approximately 3 to 4-inches of secondary consolidation spanning the design life of the structure. The effects of the secondary consolidation settlement can be minimized by using frequent vertical slip joints in the wall facing panels, and shorter parapet segments, or using a larger temporary surcharge.

A comprehensive instrumentation plan will be required during the construction of the temporary embankment to monitor the settlement and pore pressures of the foundation soil and surrounding structures. The instrumentation plan will consist of settlement monitoring plates along the preloading embankment, settlement monitoring points on existing structures in close proximity to the temporary embankment and the installation of piezometers to monitor the pore-water pressure developed within the foundation soil during the construction of the embankment. Inclinometers should also be used to verify there is no slope or foundations failures developing.

Advantages:

- Lower cost compared to Deep Mixing Method
- Strength and stiffness gain of foundation soil due to preloading

Disadvantages:

- Advanced contract for the preloading or significant risk of increased construction duration if advanced contract is not used.
- Difficult to accurately predict time rate of settlement and shear strength gain due to surcharging making the length of the quarantine period difficult to estimate. Quarantine time may need to be extended if settlement is not complete or leveled off. Risk of construction delays
- Downdrag and lateral spreading may still be an issue
- EPS embankment with unconventional flexible connections, and moment slab will be used
- Deeper undercutting for the EPS than for DMM
- Dewatering and permitting issues in the river to install and remove temporary surcharge
- Significant instrumentation required

5.2.2.5 Deep Mixing Method with Mechanically Stabilized Earth (MSE) Walls

The DMM design was based on 4-ft diameter soil mix columns with a shear wall arrangement perpendicular to the face of the ramp walls and isolated columns in the interior of the approach embankment. The soil mix columns extend through the soft soil, Stratum Ia, into the medium dense to dense soil, Stratum Ib, underlying Stratum Ia. Table 5.9 summarizes the details of the DMM arrangement. The 28 day UCC strength typically ranges for 100 to 300-psi and depends on the contractor’s means and methods of mixing as well as the reaction of the cement used with the in situ soils.

<table>
<tr><th colspan="8">Table 5.9 –Summary of Deep Mixing Method Parameters</th></tr>
<tr><td colspan="4">28 day (UCC)</td><td colspan="4">120-psi</td></tr>
<tr><td colspan="4">Soil Mix Column Diameter</td><td colspan="4">4-ft</td></tr>
<tr><td colspan="4">Shear Wall Overlap Ratio</td><td colspan="4">0.3 (1.2ft overlap for 4ft diameter columns)</td></tr>
<tr><th rowspan="2">Location</th><th rowspan="2">Start</th><th rowspan="2">End</th><th colspan="3">Shear Wall</th><th colspan="2">Isolated Column</th></tr>
<tr><th>Length (ft)</th><th>C-C Spacing</th><th>Area Ratio</th><th>C-C Spacing</th><th>Area Ratio</th></tr>
<tr><td rowspan="4">East Approach Ramp</td><td>STA 440+70</td><td>STA 441+00</td><td colspan="5">Deep Mix Columns in rectangular grid or lattice pattern</td></tr>
<tr><td>STA 441+00</td><td>STA 441+50</td><td>26.4-ft</td><td>9.0-ft</td><td>0.40</td><td>7.0-ft</td><td>0.25</td></tr>
<tr><td>STA 441+50</td><td>STA 442+50</td><td>26.4-ft</td><td>10.0-ft</td><td>0.36</td><td>7.0-ft</td><td>0.25</td></tr>
<tr><td>STA 442+50</td><td>STA 443+50</td><td>23.6-ft</td><td>12.0-ft</td><td>0.30</td><td>7.0-ft</td><td>0.25</td></tr>
</table>

The approach ramp will consist of conventional MSE walls with moment slabs and barriers on top. MSE wall typically consists of facing, such as segmental precast units, dry block concrete or CIP concrete facing units connected to horizontal steel strips, bars or geosynthetic that create a reinforced soil mass. The reinforcement is typically placed in horizontal layers between successive layers of granular backfill. A free draining, low-plasticity backfill is required to provide adequate performance of the wall.

The design of MSE for the retaining walls for internal stability will be the Contractor's responsibility and will need to be designed by a Professional Engineer licensed in the State of Delaware and reviewed by the Engineer. Minimum reinforcement length should be designed to satisfy external and global stability.

Advantages

- Soil improvement will transfer the load from the embankment to stiffer and denser foundation soil below the soft soils as well as improve the soft foundation soils
- Soil improvement will limit the settlement of the foundation soil
- Soil improvement will limit downdrag and lateral spreading on the foundation drilled shafts for the bridge abutment
- The approaches can consist of conventional MSE wall with moment slab and barriers on top. No need for unusual design elements such as flexible connection for the EPS approaches
- Reduced depth of excavation below existing ground surface for the construction of the bridging layer (3-ft). Least amount of undercutting.
- Spoils from soil mixing will be cement treated soils and can be used as embankment fill

Disadvantages

- Higher Cost
- Area required to stock pile soil mix spoils

5.2.2.6 Back Span

Additional bridge spans may be added to move the bridge approach further away from the river. The additional span will also reduce the maximum height of the approach ramp retaining walls. DelDOT requires a 4-ft clearance below the superstructure for inspection purposes. We evaluated three 100-ft back spans from the original abutment location. The approach ramps may be constructed using one of the three options listed above.

Advantages

- The reduced height of the approach ramp may reduce one preloading stage or a significant volume of EPS.
- The additional distance from the river will help in the dewatering process during construction.

Disadvantages

- Extra cost for the construction of the additional span.
- Cost for the inspection and maintenance of the additional span throughout the life of the bridge.

5.2.2.7 Stone Columns/Densified Aggregate Piers

Stone columns were evaluated as a means of supporting the approach MSE walls. However, treatment of in-situ soils with stone columns will not significantly reduce settlement or improve stability enough to comply with design criteria without a large replacement ratio. The East approach is significantly taller than West approach with a thicker soft stratum and will result is excessive settlement even with the construction of the stone columns. Excavation for columns will require large amounts of potentially contaminated soil and water to be treated/transported/disposed of.

5.3 BRIDGE FOUNDATION RECOMMENDATIONS

Bridge foundation recommendations are based on the Final structural drawings, the results of the subsurface exploration, and our experience in the area.

5.3.1 Abutment Foundations

We recommend 48-inch diameter drilled shafts be used for the support the abutments and wingwalls for the bridge.

Table 5.10 summarizes the nominal resistance and estimated drilled shaft lengths for the abutments. The west approach ramp to the bridge is designed for full load compensation using EPS and will not induce downdrag loads on to the drilled shaft foundations. Ground improvements using Deep Mixing Method (DMM) will be performed along the east approach ramp to increase the strength and stiffness of the foundation soil. The DMM will transfer the embankment load through the soft soils into the underlying competent foundation soils and will not induce downdrag loads on to the drilled shaft foundations. These drilled shafts will rely on skin friction from Strata III and IV and tip resistance in Stratum IV. The drilled shafts were designed for strain compatibility with the skin resistance and tip resistance normalized for a 0.5-inch settlement of the drilled shaft.

Table 5.10 – Abutment Drilled Shafts (48-inch)				
Location	**Factored Resistance, R_F (kips)**	**Nominal Resistance, R_n (kips)**	**Length from BOF (ft)**	**Estimated Tip Elevation**
Abutment A	1107	1582	121	-120
Abutment B	924	1319	116	-117
Resistance Factor $Ø_{STAT}$ = 0.70 (One OCL or SNLT)				

Factored resistances will be based upon a resistance factor ($Ø_{STAT}$) of 0.70, assuming a Osterberg Cell Test (OCT) or Statnamic Load Test (SNLT) for at least one drilled shaft at one of the abutments is performed, and that quality control of the remaining drilled shaft is calibrated based on the testing results. These load tests will verify the Contractor's construction techniques and the Engineer's design assumptions. The drilled shafts are spaced more than three diameters apart and a group efficiency of 1.0 was used for the design.

Temporary casing may be required for the construction of the drilled shaft. Any permanent casing left in place for the construction of the drilled shafts should not extend below EL -59 and EL -60 for Abutments A and B, respectively. Otherwise, the shaft tip may need to be lowered or the design modified to use a strain-compatible amount of tip resistance.

5.3.2 Pier Foundations

We recommend each bridge pier be supported on three 72-inch diameter drilled shafts.

Table 5.11 summarizes the nominal resistance and estimated drilled shaft lengths for the piers. These shafts rely on skin friction from Strata III and IV and tip resistance in Strata IV and V. One boring in the area of Pier 2 (Boring P2-2) did not encounter competent rock; therefore, even though the other shafts will extend to rock (Strata V), the design assumes tip resistance from Strata IV for all shafts. The drilled shafts were designed for strain compatibility with the skin resistance and tip resistance normalized for a 0.5-inch settlement of the drilled shaft.

Table 5.11 – Pier Drilled Shafts (72-inch Diameter)					
Location	**Factored Resistance, R_F (kips)**	**Nominal Resistance, R_N (kips)**	**Length from BOF (ft)**	**Estimated Tip Elevation**	**Alternate Design Rock Socket Length (ft)**
Pier 1	2263	3233	135.5	EL-140	10.0
Pier 2	2240	3200	135.5	EL-140	15.0
Resistance Factor Ø = 0.70 (One OCL or SNLT)					

Factored resistances will be based upon a resistance factor (Ø) of 0.70 assuming one OCL or SNLT is performed and that quality control of the remaining drilled shaft is calibrated based on the testing results. These load tests will verify the Contractor's construction techniques and the Engineer's design assumptions. The drilled shafts are spaced more than three diameters apart and a group efficiency of 1.0 was used for the design.

The construction of the drilled shaft will require the use of permanent casing through the water into the river bed. The permanent casing should extend a minimum of 5-ft into Stratum III (Residual Soil). The axial design of the drilled shaft was performed neglecting the skin resistance along the length of the permanent casing. The permanent casing used for the construction of the drilled shafts should not extend below EL -79 and EL-75 for Pier1 and Pier 2, respectively. Otherwise, the shaft tip will need to be lowered.

If sound bedrock is encountered at a shallow depth the shafts should extend a minimum of 10-ft and 15-ft into sound bedrock for Pier 1 and Pier 2, respectively. Sound bedrock is defined as bedrock with a recovery of at least 90%. It is recommended that prior to the installation of the drilled shafts, the Contractor conduct probe holes using either air track drilling or other testing methods at each of the drilled shaft location to verify the depth of sound bedrock. The length of the rock socket is defined as the length of excavation through rock that cannot be drilled with conventional earth or rock augers and/or underreaming tools and requires the use of special rock core barrels, air tools, and/or methods of hand excavation. Auger refusal is defined as drilling advancement of less than 2-inches in 5 continuous minutes for a 72-inch diameter rock auger with carbide teeth powered by a drilling machine applying a minimum crowd of 50,000-lb while turning the auger.

5.3.3 Design for Lateral Loads

Horizontal movement induced by lateral loads were evaluated for the Service Limit State load combinations using the software application ALLPile that uses nonlinear p-y soil responses to model responses to lateral loads.

Table 5.12 summarizes the estimated horizontal deflection and point of fixity under anticipated maximum service load combination for the pier drilled shafts assuming the 500-year scour has occurred.

Table 5.12 – Summary of Lateral Deflection of Drilled Shafts

Location	Diameter	Estimated Horizontal Deflection at BOF	Point of Fixity from BOF
Abutment A	48-inches	0.5-inches	36.7-ft
Pier 1	72-inches	0.9-inches	51.7-ft
Pier 2	72-inches	1.0-inches	47.1-ft
Abutment B	48-inches	1.1-inches	41.3-ft

BOF – Bottom of Footing
Note: Horizontal Deflection is at BOF Elevation

5.3.4 Drilled Shaft Construction and Monitoring Recommendations

We recommend that the installation of the drilled shafts be monitored by a Geotechnical Engineer or Engineering Geologist. The installation monitoring should be supervised by a Geotechnical Engineer licensed in the State of Delaware. During the installation of the drilled shafts, the depth of embedment, the diameter of the shafts, and appropriateness of the bearing materials should be verified.

A temporary protective steel casing may be required to maintain an open excavation for the abutment shafts, due to high groundwater and sandy soil. This casing can be extracted as the concreting operation progresses. A permanent steel casing will be required for the construction of the drilled shaft through the water for the pier foundations. The casing should extend a minimum 10-ft into Stratum III (Residual Soil) or the elevations stated in Section 5.3.2 of this report.

Before concrete placement commences, the bottom of the shaft excavation should be cleaned out using procedures such as airlifts, and video monitoring should be used to verify the removal of loose material.

Due to the estimated groundwater level, we recommend that the concrete be placed with a tremie or other non-free fall techniques to control concrete placement. The placement of concrete in the cased portion of the drilled shaft should proceed until the concrete level is above the external fluid level and should be maintained above this level throughout casing removal. Free fall of concrete shall not be allowed.

Appropriate testing using nondestructive techniques, such as downhole tests conducted in access tubes including cross-hole acoustic tests, backscatter gamma ray or sonic echo tests or thermal testing should be used on all drilled shafts to confirm that the shaft has been formed adequately. Cross-hole Sonic Logging (CSL) is a nondestructive technique used to verify the integrity of all the drilled shafts after the concrete has cured. CSL is used to determine the soundness of concrete within the drilled shaft inside the rebar cage. One tube per foot diameter will be installed in the drilled shaft tied to the interior of the rebar cage for CSL testing. CSL testing detects defects such as soil intrusions, necking, sand lenses, and voids within the foundation concrete. Where defects exists CSL method will determine the extent, nature, depth and lateral location of the defects so that remedial measures can be implemented. If anomalies are detected it may be necessary to core the suspect areas. Coring through the drilled shaft will be performed using a diamond core bit to retrieve samples to confirm the CSL testing results and for unconfined compression tests. The causes of the defects if present should be investigated to avoid installation of additional defective shafts during the completion of the bridge foundations and the suspect shaft will need to be repaired.

5.3.5 Excavation Difficulties

As noted previously, some of the borings encountered rock, fractured and broken rock, ledge rock, and weak seams of weathered rock and soil. These materials typically cannot be excavated by conventional methods and the excavation of the drilled shaft may require special rock augers, downhole hammers, core barrels, air tools, blasting, or hand excavation to excavate through these materials. Excavation difficulties will be affected by remnant jointing, bedding, and type of excavation equipment used. Due to the nature and weathered of the parent rock and the limited exploration in the approximate the depth of auger refusal and top of rock are only approximations for the foundations. A slow drilling rate should be anticipated where boulders or other obstructions are encountered.

Drilled shaft construction should be monitored on a full time basis by a Geotechnical Engineer or by an Engineering Geologist under the supervision of a Geotechnical Engineer licensed in the State of Delaware to document the occurrence or absence of obstructions and verify that the construction is performed in accordance with the specifications.

The presence of boulder-sized rock fragments and ledge rock interbedded with softer weathered seams will create difficulty in excavating the drilled shafts. There is an increased risk of frequent equipment switch-outs from rock removal equipment to soil augers and back again. It should be noted that due to highly variable soil conditions obstructions should be anticipated in all of the drilled shaft excavations.

5.3.6 Static Load Test (SLT) for Drilled Shafts

Static Load Testing (SLT) is recommended to verify the axial capacity of the drilled shafts. We recommend one SLT at Pier 2 and one SLT at Abutment A. The SLT at Pier 2 should be performed on the drilled shaft at the south end of the pile cap located closest to boring P2-2. Boring P2-2 did not encounter competent bedrock. A higher resistance factor of 0.7 can be used with the verification of the axial capacity of the drilled shafts using static load test. The axial capacity of the drilled shafts can be confirmed using either the Osterberg Cell Test (O-Cell or OCL) or the Statnamic test (SNLT).

The Osterberg Cell is a hydraulically driven, high capacity, sacrificial loading device installed within the drilled shaft. Working in two directions, upward against side-shear and downward against end-bearing, the Osterberg Cell automatically separates the resistance parameters. The Osterberg Cell derives all reaction from the soil and rock system. End bearing provides reaction for skin friction portion of the load test and skin friction provides reaction for the end bearing portion of the test. The Osterberg Cell will be specially instrumented to allow direct measurement of the expansion so with compression and top of drilled shaft measurements the downward end bearing movement and upward skin friction movements are known.

Statnamic testing works by accelerating a mass upward that in turn imparts a load onto the deep foundation below the Statnamic device. The load is applied and removed smoothly resulting in load application of 100 to 200 milliseconds. During the loading sequence the load applied to the test shaft is monitored by a calibrated load cell incorporated in the base of the combustion piston. A remote laser reference source that falls on a photovoltaic cell incorporated in the piston will be used to measure the foundation settlement. The equivalent static load-settlement curve will be derived from the Statnamic data. The unloading point method (UPM) analysis should be performed to obtain the equivalent static response.

5.4 UNDERPASS FOUNDATION RECOMMENDATIONS

Based on the foundation loads described in Section 2 of this report, the results of the subsurface exploration and our experience in this area, we recommend the underpass be supported on 48-inch diameter drilled shaft foundations.

Table 5.13 summarizes the nominal resistance and estimated drilled shaft lengths for the underpass. These drilled shafts will rely on skin friction from Strata III and IV and tip resistance from Stratum IV. The drilled shafts were designed for strain compatibility with the skin resistance and tip resistance normalized for a 0.5-inch settlement of the drilled shaft.

Table 5.13 – Underpass Drilled Shafts (48-inch)

Location	Factored Resistance, R_F (kips)	Nominal Resistance, R_n (kips)	Length from BOF (ft)	Estimated Tip Elevation
East Wall	979	1399	121	-120
West Wall	592	846	119	-115
Resistance Factor $Ø_{STAT}$ = 0.70 (One OCL or SNLT)				

Factored resistances will be based upon a resistance factor ($Ø_{STAT}$) of 0.70, assuming an OCL or SNLT for at least one drilled shaft at one of the abutments is performed, and that quality control of the remaining drilled shaft is calibrated based on the testing results. These load tests will verify the Contractor’s construction techniques and the Engineer’s design assumptions. The drilled shafts are spaced more than three diameters apart and a group efficiency of 1.0 was used for the design.

Temporary casing may be required for the construction of the drilled shaft. Any permanent casing left in place for the construction of the drilled shafts should not extend below EL -59. Otherwise, the shaft tip may need to be lowered or the design modified to use a strain-compatible amount of tip resistance.

5.5 STAIRS FOUNDATION RECOMMENDATIONS

Based on the foundation loads described in Section 2 of this report, the results of the subsurface exploration and our experience in this area, we recommend that the proposed stairs be supported on 36-inch diameter drilled shaft foundation.

Table 5.14 summarizes the nominal resistance and estimated drilled shaft lengths for the stairs. These drilled shafts will rely on skin friction only from Strata II, III and IV.

Table 5.14 – Stairs Drilled Shafts (36-inch)

Location	Factored Resistance, R_F (kips)	Nominal Resistance, R_n (kips)	Length from BOF (ft)	Estimated Tip Elevation
Stairs	229	458	86	EL -78
Resistance Factor Ø = 0.50 (Static Analysis)				

5.6 BRIDGE APPROACH RECOMMENDATIONS

5.6.1 West Approach Ramp

Based on the proposed ramp height, the subsurface conditions, and our experience in the area we recommend that the west side ramp be constructed of Expanded Poly-Styrene (EPS) with conventional abutment and wingwalls supported on drilled shafts. The west approach will consist of approximately 145-ft of EPS Retaining wall from STA 434+00 to STA 435+45 and 21-ft long drilled shaft supported CIP concrete wingwalls from STA 435+80 to STA 436+01. The concrete arch underpass will be located behind the abutment from STA 435+45 to STA 435+80. The approach ramp west of STA 434+00 will consist of an embankment constructed of Type F – Common Borrow, and it will have 2(H):1(V) side slopes.

Light weight flowable fill should be used between the abutment, CIP wingwalls and the underpass to minimize the lateral loads to the abutment and wingwalls. To eliminate any additional settlement and resulting downdrag loads on the drilled shafts the embankment fill between the abutment and the underpass will be supported on a structural slab spanning the abutment and underpass and supported on the drilled shaft foundation.

To eliminate any additional settlement and resulting downdrag loads on the drilled shafts west of the underpass a fully compensated embankment should be constructed by undercutting the foundation soils to a depth of 4.5-ft below the existing ground surface. We recommend the existing soils be undercut to this depth to eliminate downdrag on the drilled shafts and settlements of the backfill and drilled shafts even though there will be additional costs associated with handling contaminated soil and groundwater.

The construction of a conventional abutment and wingwalls supported on drilled shafts will provide greater slope stability towards the river. The drilled shaft foundation is designed for the anticipated scour depth. This option will provide an economical design at moderate risk. The construction of an EPS bridge approach will require a shorter construction schedule compared to the other options considered.

Lowering the bottom of footing of the CIP wingwalls below the scour elevation shown in Table 2.3 will require approximately 9-ft of excavation below the existing ground surface and 2.5-ft below the ground water elevation. The soil in the proposed wall location contains elevated concentration of metals (lead and arsenic) and petroleum and will need to be treated and disposed at additional cost. The excavation below ground water table will also require extensive dewatering.

5.6.2 East Side Ramp Walls

Based on the proposed ramp height, the subsurface condition, and our experience in the area we recommend that the east side ramp be constructed of MSE walls supported on DMM columns with a conventional abutment and wingwalls supported on drilled shafts. The soil improvement using DMM will extend from Abutment B at approximately STA 440+70 to STA 443+50. The east approach will consist of 30-ft of drilled shaft supported CIP concrete wingwalls from STA 440+70 to STA 441+00 and approximately 200-ft of MSE Retaining wall from STA 441+00 to STA 443+00. The embankment east of STA 443+00 will be constructed of Type F – Common Borrow.

The DMM design was evaluated for the failure modes and Factor of Safety listed in Table 5.15 of this report (Ref: FHWA Design Manual: Deep Mixing for Embankment and Foundation Support). Verification testing performed during construction will verify the design assumptions and the design can be revised for areas where needed.

Table 5.15 – Summary of DMM Failure Modes and Factors of Safety	
Failure Modes	**Factor of Safety**
Slope Stability Failure	1.5
Combined overturning and bearing capacity of the deep mixed shear walls	1.3
Crushing of the deep mixed ground at the toe of the deep mixed zone	1.3
Shearing on vertical planes through the deep mixed zone	1.3
Soil extrusion through deep mixed shear walls	1.3

The DMM design was based on 4-ft diameter soil mix columns with a shear wall arrangement perpendicular to the face of the ramp walls and isolated columns in the interior of the approach embankment. The plan arrangement of the DMM columns in shown in Figure A-5. The soil mix columns extend through the soft soil, Stratum Ia, into the medium dense to dense soil, Stratum Ib, underlying Stratum Ia. The preliminary design consists of 552 soil mix columns along the east approach ramp. The load from the approach embankment will be transferred to the DMM columns through a 3.5-ft thick geosynthetic reinforced Graded Aggregate Base (GAB) load transfer platform. The load transfer platform will be constructed immediately above the columns to help transfer the load and prevent a “bearing capacity” type of failure above the columns. The load transfer platform minimizes differential settlement for lower height embankments.

Using DMM will assist in constructability of the conventional abutment and wingwalls supported on drilled shafts and will provide greater slope stability towards the river. The drilled shafts can be designed for the anticipated scour depth. To minimize the depth of excavation at the abutment a scour protection system consisting of a cantilever sheet pile wall can be constructed in front of the abutment and wingwalls. This option will provide a more economical design with lower risk and a shorter construction period than the other options considered.

Lowering the bottom of footing of the CIP wingwalls to below the 500-year scour elevation as shown in Table 2.3 will require approximately 10-ft of excavation below the existing ground surface and below the ground water elevation. The soil in the proposed wall location contains elevated concentration of metals (lead and arsenic) and petroleum and will need to be treated and disposed at additional cost or stockpiled, dewatered and re-used onsite. The excavation below ground water table will also require extensive dewatering.

Because of this, we recommend providing scour protection along the abutment and wingwalls so that excavation for the wall footings can be reduced to that required for the construction of a load transfer platform and pile cap. This will reduce the cost required for disposing of the contaminated and wet soil and water collected during dewatering of the excavation. This scour protection will consist of a sheet pile system used to aid in dewatering and support of excavation during construction and then left in place as scour protection. If the soil is re-used on site, then the soils will not need to be treated but room will be required for stock piling and dewatering the soils on site.

5.6.2.1 MSE Wall

The following sections are general recommendations for construction of the MSE retaining walls. The detailed internal and external stability design of the MSE walls is the Contractor's responsibility and will need to be designed by a Professional Engineer licensed in the State of Delaware and reviewed by the Engineer. For our analysis, we evaluated the global and external stabilities (bearing capacity, sliding, and overturning) and settlements to determine the suitability of MSE construction for this project.

Bearing Resistance

The nominal bearing resistance, Meyerhof stress, and eccentricity (e) were estimated using a software program entitled MSEW, a design and analysis software for mechanically stabilized earth walls, and with manual hand calculations. The factored bearing resistance was estimated using the following equation:

$$q_r = \phi_b q_n$$

Where: q_r – Factored Bearing Resistance

ϕ_b – Bearing Resistance Factor from AASHTO (Table 11.5.6-1)

ϕ_b – MSE Walls = 0.65

q_n – Nominal Bearing Resistance

Proper construction procedures should be used to maintain the bearing qualities of the footing excavations. Foundations and excavations should be protected from the detrimental effects of precipitation, seepage, surface run off, or frost. The shear strength of the foundation soil was based on the replacement area ratio of the DMM. The minimum reinforcement length to height ratio for the retaining wall is L/H=0.7.

Corrosion Protection

The reinforcing straps for the MSE wall will be embedded in No. 57 Stone and not in situ materials. As indicated in FHWA NHI-00-044 the retaining wall backfill material should meet certain electrochemical properties. Table 5.16 below details the limits of electrochemical properties and the corresponding test method.

Table 5.16 - Limits of Electrochemical Properties for Backfill

Property	Criteria	Test Method
Resistivity	Greater than 3,000 ohm-cm	AASHTO T-288-91
pH	5 to 10	AASHTO T-289-91
Chlorides	Less than 100 PPM	AASHTO T-291-91
Sulfates	Less than 200 PPM	AASHTO T-290-91
Organic Content	1% max	AASHTO T-267-86

Fill for Reinforcement Zones

Fill in the reinforcement zone should consist of No. 57 stone. No. 57 stone placed in the reinforcement zone should be in accordance with Section 813 – Grading Requirements Minimum and Maximum Percent Passing, Delaware Department of Transportation; ***Specifications for Road and Bridge Construction***, dated August 2001 with supplements.

The materials should be substantially free of shale or other soft, poor-durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles, measured in accordance with AASHTO T-104, or a sodium sulfate loss of less than 15 percent after five cycles determined in accordance with AASHTO T-104.

Light weight walk behind compaction equipment may be required near the wall face to attain the proper degree of compaction without overstressing connections or the facing panels. Extra care should be given to avoid damaging the wall due to heavier loads produced by larger construction equipment.

Onsite soil (Type F borrow) may be used to construct the remainder of the embankment behind the MSE. This should be placed and compacted in accordance with in accordance Section 202 – Excavation and Embankment, Delaware Department of Transportation; ***Specifications for Road and Bridge Construction***, dated August 2001 with supplements.

Reinforcement Length and Global Stability

A resistance factor of 0.65, approximately a minimum Factor of Safety (FS) of 1.5, was used to evaluate global stability. The reinforcement length for all retaining walls should be a minimum of 0.7H, where H is the height of the retaining wall from the top of the leveling pad to the ground surface above the wall, unless otherwise noted below. The minimum length of reinforcement regardless of the wall height should be 8-ft.

The global stability was evaluated using the following software program:

- Slope/W is a slope stability analysis program that evaluates the stability of slopes using limit equilibrium methods. The stability of a slope can be evaluated using either deterministic or probabilistic input parameters. For this project, the Morgenstern-Price method was used.

5.6.2.2 Instrumentation Monitoring

A construction monitoring program consisting of settlement plates should be implemented to monitor the settlement below the east approach ramp as construction progresses. Table 5.17 summarizes the recommended locations of the settlement plates.

Table 5.17 – East Approach Ramp Instrument Locations

Instrument	Station	Center	Edge
Settlement Plate	STA 440+75	1	----
	STA 441+50	1	2 (one on each edge)
	STA 442+50	1	----

The settlement monitoring plates should be read, weekly for three weeks prior to mobilizing construction equipment to the project site, at least weekly during construction and 30-days after completion of the filling operations, and bimonthly for a period of approximately 6-months. This schedule maybe modified once construction starts and may be relaxed if little movement is noticed. The monitoring points should be established to an accuracy of at least 0.02-inch in elevation.

The selection of the monitoring points should be approved by the Engineer. Daily observations should be made and documented to determine if any surficial signs of distress are evident. During construction frequency of the monitoring program may be adjusted by the Contractor with the approval of the Engineer. The instrumentation data should be presented in graphical and tabular formats. The instrumentation data should be provided to the Engineer within 24-hours or one business day after each reading.

5.6.3 Sheet Pile Wall for Scour Protection at East Abutment

Based on the proposed bottom of footing elevation of Abutment B at EL -1 and the 500-year scour elevation at EL-5 we recommend a cantilever sheet pile wall for scour protection in front of the abutment and around the wingwalls. The sheet piles should run along the face of the abutment with a 40-ft return on the north side and 30-ft return on the south side. The top of the sheet piles should be located at EL+3. The weight of the backfill between the wing walls and abutment will be transferred to the drilled shaft foundations using a structural slab. The sheet piles were designed for a maximum exposed height of 8-ft (EL +3 to EL -5). The cantilever sheet pile wall was designed to limit the lateral deflection to less than 1-inch. Table 5.18 summarizes the size, embedment depth, and estimated deflection of the sheet pile wall.

Table 5.18 – Summary of Sheet Pile Wall

Location	Design Height	Sheet Pile Size	Embedment Depth	Maximum Deflection
Abutment B	2-ft Additional Surcharge	PZ 27	21-ft (Tip EL-26)	1.0-inches

5.7 ROADWAY EMBANKMENT (EAST OF STA 444+00)

The roadway embankment east of STA 444+00 will be constructed using Type F – Common Borrow. The maximum height of embankment fill will be approximately 7-ft above the existing ground surface at approximately STA 448+50. The side slopes of the roadway embankment will be approximately 3(H):1(V). We evaluated the stability of the embankment slope and the anticipated long term settlement of the embankment.

5.7.1 Settlement

Due to the presence of very weak foundation soil the construction of the roadway embankment will result in total settlement in excess of 9-inches. Table 5.19 summarizes the estimated settlement of the roadway embankment.

Table 5.19 – Total Settlement of Roadway Embankment

Location	Loading Condition	Immediate Settlement	100% Consolidation Settlement
STA 448+50	Proposed Grade	1.1-inches	8.2-inches
	2-ft Additional Surcharge	1.4-inches	9.9-inches

We estimate that the long term settlement will take approximately 2 years. To minimize the effect of the long term settlement we recommend the roadway embankment be constructed with a 2-ft additional surcharge above the proposed grade and quarantined for a minimum time period of 5-months. The additional 2-ft surcharge will accelerate the long term settlement of the embankment. We calculated approximately 71% of the consolidation settlement (7-inches) will be complete after the 5 month quarantine period. The estimated remainder of the long term settlement will be approximately 1.2 inches. We estimated it will take approximately 2-years for 100% completion of the consolidation settlement.

5.7.2 Slope Stability

The stability of the embankment slopes were evaluated to determine if the proposed embankment with the 2-ft surcharge, i.e. 9-ft total height can be constructed in a single phase. Table 5.20 summarizes the Factor of Safety for global stability for the proposed embankment and the additional 2-ft surcharge.

Table 5.20 – Summary of Factor of Safety for Global Stability

Location	Embankment	Maximum Height	Factor of Safety
STA 448+50	Proposed Grade	7.0-ft	1.427
	2-ft Additional Surcharge	9.0-ft	1.160

The factor of safety for the temporary condition with the 2-ft surcharge is higher than 1.1 and the permanent condition for the proposed grade is higher than the required factor of safety of 1.3 for roadway embankments.

5.7.3 Instrumentation Monitoring

A construction monitoring program consisting of settlement plates, and piezometers should be implemented to monitor the settlement and pore pressure below the embankment as construction progresses. Table 5.21 summarizes the recommended locations of the settlement plates and piezometers. In addition, surface monitoring points should be installed on the Load Distribution slab and the leveling pads of the MSE and EPS facing panels.

Table 5.21 – Instrument Locations

Instrument	Station	Center	Edge
Settlement Plate	STA 447+50	1	----
	STA 448+50	1	2 (one on each edge)
	STA 449+50	1	----
Vibrating Wire Piezometer in Stratum Ia	STA 447+50	1	----
	STA 448+50	3 (in vertical array)	2 (one on each edge)
	STA 449+50	1	----

The settlement monitoring plates and piezometers should be read, weekly prior to mobilizing construction equipment to the project site, at least weekly during construction and 30-days after completion of the filling operations, and bimonthly for a period of approximately 6-months. This schedule maybe modified once construction starts and may be relaxed if little movement is noticed. The monitoring points should be established to an accuracy of at least 0.02-inch in elevation.

The selection of the monitoring points should be approved by the Engineer. Daily observations should be made and documented to determine if any surficial signs of distress are evident. During construction frequency of the monitoring program may be adjusted by the Contractor with the approval of the Engineer. The instrumentation data should be presented in graphical and tabular formats. The instrumentation data should be provided to the Engineer within 24-hours or one business day after each reading.

Settlement monitoring for the quarantine times for the embankment fill can be achieved using repeatable survey of settlement plate monitors reference to at least two permanent bench marks.

The piezometers should be installed in Stratum Ia and readings taken at least 2 weeks prior to mobilizing construction equipment to the project site to gather background data. The Threshold and Limiting levels of pore water pressure are as follows:

Threshold Value	Pore water pressure = 50% of the applied surcharge
Limiting Value	Pore water pressure = 90% of the applied surcharge

The construction of the embankment should be monitored closely with the rise in pore water pressure in Stratum Ia above 50% of the applied surcharge. The construction of the embankment should be stopped if the pore pressure in Stratum Ia rises above 90% of the applied surcharge. The construction of the embankment can be continued after the pore water pressure dissipates to below 50% of the applied surcharge.

5.8 SPECIAL CONSIDERATIONS

5.8.1 Corrosion Potential

Corrosion potential for this project is based on the corrosion and deterioration criteria set forth in AASHTO, Section 10.7.5. For this project, the following applicable soil corrosion potential criteria from AASHTO, Section 10.7.5, is indicated below.

- pH less than 5.5, or
- Resistivity less than 2,000 ohm-cm, or
- Sulfate concentrations greater than 1,000-ppm.

The results of the Corrosion Potential Testing are summarized in Table 3.7. The results indicated that the sulfides content in the soils were negligible. The pH measurement of two soil samples on the West side were lower than 5.5 indicating acidic soils. Soils that are extremely acidic are generally associated with significant corrosion rates.

The resistivity test of the soil samples indicated that the foundation soils on both banks of the river have values that ranged from 470 ohm-cm to 1500 ohm-cm. Corrosion increases as resistivity decreases. The relative level of corrosiveness, commonly accepted by the engineering community as indicated by resistivity levels is shown in Table 5.22.

Table 5.22– Effect of Resistivity on Corrosion

Aggressiveness	Resistivity in ohm-cm
Very Corrosive	<700
Corrosive	700 – 2,000
Moderately Corrosive	2,000 – 5,000
Mildly Corrosive	5,000 – 10,000
Non-Corrosive	>10,000
Reference: FHWA-NHI-00-044 Corrosion/Degradation of soil reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes	

5.9 GENERAL EARTHWORK RECOMMENDATIONS

All existing structures, including all above and below ground construction, within the areas of the proposed construction should be removed prior to initiation of construction. Topsoil, other organic materials, frozen, wet, soft or loose soils, and other deleterious materials should be removed and wasted before placement of fill. These stripping operations should be performed in a manner consistent with good erosion and sediment control practices.

After stripping, areas where pavement will be placed should be proofrolled with a heavily-loaded (35-ton) dump truck or another pneumatic-tired vehicle of similar size and weight where possible. The purpose of the proofrolling is to provide surficial densification and to locate any isolated areas of soft or loose soils. Unsuitable areas should be undercut and replaced with controlled compacted fill as described in Section 5.7. A geotechnical engineer licensed in the State of Delaware or an engineering technician under the supervision of such an engineer should witness the stripping and proof rolling operations.

Stripping, clearing, grubbing and proof rolling should be performed in accordance with the Delaware Department of Transportation (DelDOT), **Standard Specification 2001**, Section 201.

5.10 FILL SELECTION, PLACEMENT AND COMPACTION

In general, existing on-site soils free from environmental contamination, building debris, frozen, organic or wet materials, consistent with the provision in Delaware Department of Transportation (DelDOT), **Standard Specification 2001**, Section 209 “Borrow”, can be reused as compacted fill as described below.

Common borrow should meet the requirements of Borrow Type F and should be in accordance with the requirements of AASHTO M 57, Modified; M 145, Modified; and M 146 and M 147, Modified. The material shall have a maximum density not less than 105-pcf as determined by ASTM D698 Method C, with liquid limit less than 50. The maximum particle size should be limited to half of the loose lift thickness.

Fill in structural areas should be placed in horizontal, eight-inch maximum loose lifts and compacted to at least 98 percent density per ASTM D698 Method C. If walk behind equipment is used it may be necessary to limit the loose lift thickness to 4-inches.

The moisture content of the fill should be properly controlled during placement. Moisture content of the fill materials should be within plus or minus 2-percent of optimum moisture content as determined by the ASTM D698 Method C moisture-density test procedure. In-place density tests should be performed by an engineering technician on a full-time basis under the supervision of a geotechnical engineer licensed in the State of Delaware to verify that the proper degree of compaction is being obtained.

5.11 DEWATERING AND DRAINAGE

All work should be constructed in a relatively dry condition. This will require the construction of a coffer dam consisting of steel sheet piling at the abutment locations. The actual dewatering scheme should be determined by the Contractor since this will have a significant effect on the construction means and methods. The bottom of footing for the abutments will be below the groundwater table encountered in the borings. The high groundwater table will cause construction difficulties along with high and low tides of up to 6.5-ft. The bottom of footing for the ramp retaining walls will be near or below the groundwater table encountered in the borings.

The Contractor should be prepared to dewater any groundwater, surface runoff, or water collected after a rain event. It is likely that dewatering can be accomplished by ditching and pumping from sumps in the retaining wall locations. The water from the dewatering operation should be collected, tested for contamination and treated before it is allowed to flow into any watercourse, adjacent drainage way, or over land.

Adequate drainage should be provided at the site to minimize any increase in moisture content of the foundation soils. All run-offs from adjacent areas should be diverted away from the bridge, retaining walls, and excavations to prevent ponding of water. The site drainage should also be such that the run-off onto adjacent properties is controlled properly. Sediment laden water should not be allowed to flow into any watercourse, adjacent drainage way, or over land without first filtering it through an approved desilting device.

6 BASIS OF RECOMMENDATIONS

This report has been prepared to present the geotechnical conditions at the site, the recommended method of founding the proposed construction. The opinions, conclusions and recommendations contained in this report are based upon our professional judgment and generally accepted principles of geotechnical engineering. Inherent to these are the assumptions that the earthwork and foundation construction should be monitored and tested by an engineering technician acting under the guidance of a geotechnical engineer licensed in the State of Delaware.

These analyses and recommendations are, of necessity, based on the information available at the time of the actual writing of the report and on the site conditions, surface and subsurface, that existed at the time the exploratory borings were drilled. Further, assumptions have been made regarding the limited exploratory borings, in relation to both the lateral extent of the site conditions and to the depth.

The nature and extent of variations between borings may not become evident until construction. If variations from the anticipated conditions are encountered, it may be necessary to revise the recommendations in this report.

Our professional services have been performed in accordance with generally accepted engineering principles and practices; no other warranty, expressed or implied, is made. RK&K assumes no responsibility for interpretations made by others on the work performed by RK&K.

We recommend that this report be made available in its entirety to contractors for informational purposes only. The boring logs and laboratory test data contained in this report represent an integral part of this report and incorrect interpretation of the data may occur if the attachments are separated from the text. The project plans or specifications should include the following note:

> *A geotechnical report has been prepared for this project by Rummel, Klepper & Kahl, LLP. This report is for informational purposes only and shall not be considered as part of the contract documents. The opinions and conclusions of RK&K represent our interpretation of the subsurface conditions and the planned construction at the time of the report preparation. The data in this report may not be adequate for contractors estimating purposes.*